CONCERNING THE DESIGN OF STEEL STRUCTURES.

A THESIS

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by

David S.Stewart, B.Sc.(Hons.) (Eng.,Lond.)

Assoc.M.Inst.C.E., A.R.T.C., F.R.S.E.

CONTENTS

Chapter XI

THE DESIGN OF A JOIST AND CHANNEL CRANE GANTRY GIRDER

CONTENTS OF VOLUME II

CHAPTER IV

COLUMNS—*Continued*

TYPES: LACING, ETC., NON-AXIALLY LOADED COLUMNS AND THEIR FOUNDATIONS

CHAPTER V

THE DESIGN OF A 55-FT. SPAN ROOF TRUSS

CHAPTER VI

ROOF TRUSSES, PORTAL TRUSSES AND WORKSHOPS

CHAPTER VII

THE DESIGN OF A LATTICE GIRDER FOOT BRIDGE

Chapter VIII

EQUIVALENT UNIFORMLY DISTRIBUTED LIVE LOAD

Chapter IX

THE DESIGN OF A 70-FT. SPAN THROUGH RAILWAY BRIDGE

ABBREVIATIONS AND SYMBOLS

The following symbols are also explained throughout the text

A	Area of one flange of a plate girder.
A_c	Ditto for compression flange.
A_t	Ditto for tension flange.
B	Bearing (of a rivet).
B.M.	Bending Moment.
B.S.S.	British Standard Specification.
B.S.S. (3/18)	Ditto, part 3, article 18.
C.G.	Centre of gravity.
D	Effective depth of a girder.
D.S.	Double shear.
E	Young's modulus = stress ÷ strain.
F	Force.
G	The first moment.
H	Joist or beam.
I	Joist or beam.
I	Moment of inertia.
L	Span or length in feet.
M	Bending moment (B.M.).
M. of I.	Moment of inertia.
N.A.	Neutral axis.
P	Force, generally a pull.
Pl.	Plate.
R	Resultant.
R	Rivet value: least in S.S., D.S., or B.
R.S.J.	Rolled steel joist.
S	Total flange stress.
S.S.	Single shear.
T	Tee bars.
T	small capital, superscript, = tons.

V	Total vertical shear.
W	Web area.
W	Load or force.
Z	Section modulus.
b	Breadth of a section.
d	Diameter of a rivet or hole.
d	Depth of a rectangle.
e	Eccentricity of load in inches.
f_c	Working stress in tons/sq. in. compression.
f_t	Working stress in tons/sq. in. tension.
f_w	Working stress in tons/sq. in. shear on web pls.
f_b	Working stress in tons/sq. in. bearing on rivets.
f_{ds}	Working stress in tons/sq. in. double shear on rivets.
f_s	Working stress in tons/sq. in. single shear on rivets.
l	Span or length in inches.
p	Rivet pitch.
t	Plate thickness in inches.
v	Vertical shear per sq. inch
y	Distance to outer fibres from the N.A.
Δ	Deflection.
Δ	Triangle.
ϕ	Diameter of a rivet or hole.
$\ngtr$	Not greater than.
$\nless$	Not less than.

PRACTICAL DESIGN OF SIMPLE STEEL STRUCTURES

CHAPTER I

ROLLED SECTIONS FOR STRUCTURAL PURPOSES

Sections.—Before entering into the discussion of details a short discourse upon the sections used in constructional work may prove helpful, since successful designing depends upon many things, not least of which is the choice of suitable sections.

The white-hot and plastic ingot of steel, known as a bloom, is reduced either under a steam hammer or in a cogging mill. From there it is taken to the finishing rolls. These may be likened to huge mangles with a pair of horizontal steel or iron rollers, one mounted above the other.

Plates are formed by passing and repassing the metal through the rolls, which are made to approach each other previous to each successive rolling, until the desired thickness of metal is obtained.

In the case of other sections the cylindrical rolls have circumferential grooves or chases let into their surfaces. The metal when rolled is crushed into this chase and emerges at the other side a different shape from what it had at entry. An originally square bloom after passing through a varying set of chases finally comes forth as an angle bar or some other type of section, depending upon the outlines cut into the rolls. The finishing roll of an angle mill is given in Fig. 1.

Steel section rollers have their names cut into the rolls, and the finished section bears, in raised letters, the maker's name and sometimes the size and weight in pounds per foot length of the section. There is in addition, according to the British Standard Specification (B.S.S.), "Materials and Workmanship," a mark which enables the finished steel to be traced to the original cast.

Continental steel sections are usually cheaper than British sections, but the home article is rather more dependable, and is distinguishable from the foreign material by reason of the maker's name, which is rolled on the section. With the exception of American sections, foreign sections are usually rolled to metric dimensions. These centesimal dimensions have to be converted into inches, and on adding up dimensions the odd decimals of an inch are at times rather troublesome.

Sectional Growth.—After constant use the chases in the rolls become enlarged and the finished article has a slightly larger cross sectional area than that turned out when the rolls were new. In consequence of this steel manufacturers claim a ROLLING MARGIN of $2\frac{1}{2}$ per cent. over or under the listed weight per foot. The custom of paying the mills for the actual weights passed over the weighbridge has caused the sections to be rolled slightly on the heavy side. It is preferable that these should be over rather than under the weights listed, as the area of metal allowed for in the design is thereby assured. Any excess over the $2\frac{1}{2}$ per cent. margin is not paid for by the receiver. The flanges of a joist or the legs of an angle are

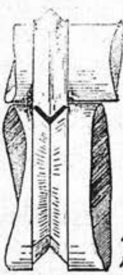


FIG. 1.
Finishing Rolls for an Angle Bar.

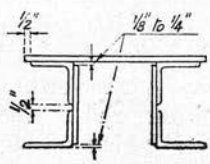


FIG. 2.

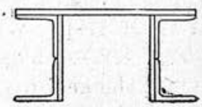


FIG. 3

often slightly wider than the listed dimensions, due to the aforementioned wearing of the rolls, and for this reason it is wise never to butt the edges of sections together when detailing a drawing.

The arrangement of the component parts of the section of Fig. 2 permits slight increases in the sizes of the individual members without departing from the general outline of the built-up section. In Fig. 3 everything is shown as butting tight, and no allowance is made for sectional growth, with the result that when actually assembled the continuity of outline is broken. Angles will project out past plates, or *vice versâ*.

PLATES (abbreviation "Pl.")

Plates may be of any size or thickness provided they do not exceed the maxima given in Table 1, Vol. III. The maker can deliver a maximum area of plate of a given thickness, but if the width of the plate is large, then the length of the plate must be correspondingly shorter, or *vice versâ*; the product of the length and the breadth must not exceed the maximum area.

Plates should always be specified as breadth by thickness by length, *e.g.*, 2 Pls. $16'' \times \frac{3}{8}'' \times 9' 0''$. The breadth, be it noted, is in inches, and not in feet and inches, as this facilitates the "running out of the weights" when estimating and diminishes the labour of collecting the plate sizes when ordering the material. In spite of the advantages accruing from this method, it is not uncommon to find the plates specified as 2 Pls. $9' 0'' \times 1' 4'' \times \frac{3}{8}''$. The beginner is prone to be slipshod in little things like this until he appreciates that these trifles when properly systematised mean a saving of time and money.

Plates which have to be planed on both long edges should be ordered $\frac{1}{4}$ in. or $\frac{3}{8}$ in. wider than the finished width. This permits $\frac{1}{8}$ in. or $\frac{3}{16}$ in. being planed off each side.

It will be observed from Table 1 that plates varying by $\frac{1}{32}$ in. are avoided in structural work, and only plates whose thicknesses are some multiple of $\frac{1}{16}$ in. are used. One can differentiate at a glance between a $\frac{5}{8}$ in. and an $\frac{11}{16}$ in. plate, but more care must be exercised when choosing between two plates whose thicknesses are $\frac{21}{32}$ in. and $\frac{11}{16}$ in. respectively.

Large plates have a tendency to arrive in the yard from the mills slightly thicker in the centre than round the edges. This seldom has any significance in practice.

Plate Extras.—Plates larger than those listed in the table are rolled, but are subject to special arrangements and prices.

Rectangular plates of $\frac{3}{8}$ in. minimum thickness are at a basis (*i.e.* lowest) price per ton.

List of Extras Charged.

Thickness.—A rising extra is charged on plates which are thicker than $1\frac{1}{2}$ in., viz., $1\frac{1}{2}$ in. to $1\frac{5}{8}$ in., over $1\frac{5}{8}$ in. to $1\frac{3}{4}$ in., and over $1\frac{3}{4}$ in. up to 2 in.

Thinness.—Plates under $\frac{3}{8}$ in. to $\frac{5}{16}$ in. thick are charged an extra, which is increased if the thickness lies between $\frac{5}{16}$ in. and $\frac{1}{4}$ in. Material which is thinner than $\frac{1}{4}$ in. is designated a sheet, and the rolling margin (of 5 per cent., *i.e.*, $2\frac{1}{2}$ per cent. under or over) is increased to 10 per cent., *i.e.*, 5 per cent. under or over.

Width.—An extra is demanded on plates 20 ft. long and upwards and under 18 in. down to 12 in. in width.

Twice the foregoing extra on plates 15 ft. long and upwards and under 12 in. in width.

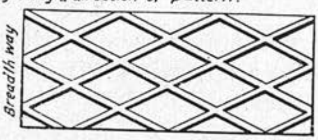
These narrow plates cannot be sheared exactly parallel-sided; the curve varies more or less according to the length and thickness and the ordinary rolling margin is not applied.

Sketch Plates.—Extras are charged on all plates whose shapes necessitate more than four cuts with the shears, *i.e.*, other than rectangular plates. If there is an extra for area, the tapered or sketch plate is charged upon the area of the parent plate from which it was cut.

Weight.—Four tons and under, free of extra, an extra for each increase of 5 cwt. over this "free" weight.

Chequered Plates are plates which have a raised diamond pattern on their surface after the manner of Fig. 4. Their thickness is generally "measured on the plain, *i.e.*, under cheque." They prove extremely useful as flooring where light loads have to be carried, such as on access gangways and machine pit covers. In addition they are easily lifted and replaced, while the chequered surface presents a foothold not offered by an ordinary plate, especially where oil is apt to be spilled.

Length way & direction of pattern.



– ADMIRALTY DIAMOND CHEQUERED PLATE –

FIG. 4.

FLATS (Symbol —)

The lower of the two rolls, through which the bar finally passes, has flanges which give the bar a definite width; *vide* Fig. 5.

If ordered to the correct width, a flat needs no planing to true it up. However, should the flat be ordered long and narrow, it will possibly not be rectangular, but will be slightly curved in its length.

Flats are slightly dearer than plates, but as their proper use saves

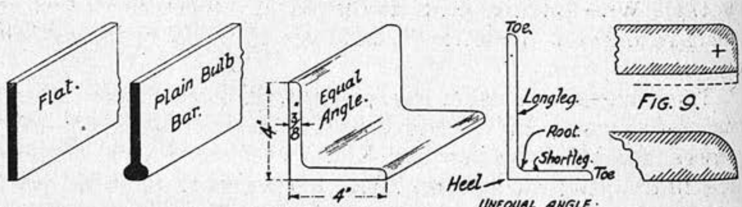


FIG. 5. FIG. 6. FIG. 7. FIG. 8. FIG. 9. FIG. 10

handling and edge planing, they probably come out cheaper. The quantity and the job on hand will settle whether they should be used or not. The corner edges of flats are often a trifle rounded and not clean-cut; they thus have the disadvantage that when piled together to form the flange of a plate girder they do not present the same perfect finish as machined plates. Their use in this case, therefore, depends upon the engineer's specification and whether the girder is exposed to near view or not. If the girder is small and only one flange plate is required, the ideal section to use is the flat. Should the flange plate require to be spliced, and if the girder is in near view, use plates, as more perfect alignment is obtainable.

Maximal Sizes to which flats are rolled are listed in Table 2, Vol. III.

Bulb Bars.—A bulb bar is a flat with a bulbous edge (Fig. 6). The bulb portion stiffens the flat should it be in compression, and the line of rivets rather far removed from the free edge of the flat. This section is more used in shipbuilding than in bridge and shop construction.

Flat Extras.—The lowest-priced flat is that which is between 10 ft. and 40 ft. in length and at least 5 in. wide, with a minimum thickness of $\frac{1}{2}$ in. The cost of flats fulfilling this condition is about 10*s*. per ton over angle basis price.

Extras Charged on Flats over Angle Basis Price.

Length.—No extra for length if between 10 ft. and 40 ft. long.

If over 40 ft. long, an extra is charged.

Rising extra charged: 10 ft. to 5 ft., under 5 ft. to 3 ft., and under 3 ft.

Width.—For flats under 5 in. wide an extra is charged.

Thickness.—Flats which are under $\frac{1}{2}$ in. thick have a rising extra according to the decrease in thickness, *i.e.*, from $\frac{1}{2}$ in. to $\frac{3}{8}$ in.; $\frac{3}{8}$ in. to $\frac{5}{16}$ in.; $\frac{5}{16}$ in. to $\frac{1}{4}$ in.

A flat may thus have three extras on it; *e.g.*, flats ordered as $4'' \times \frac{5}{16}'' \times 8'\ 0''$ from the mills would require to pay an extra for width, thickness and length. The length extra can be eliminated by ordering 16 ft. lengths and cold-sawing them at the constructional works. With the disc blade of the saw in proper order, two well-finished ends are obtained for about 2*d*., provided a straight end is required.

These extras present a somewhat formidable list, rising from 1*s*. 6*d*. to over £1 sterling per ton for different single extras. Prices are purposely not given, as they vary somewhat with the firms; and further the ruling prices can be obtained free on application to the individual steel section mills.

ANGLES (Symbols, L for equal legs and **L** for unequal legs, Figs. 7 and 8)

This particular section is probably the most useful section rolled. The thickness of each leg is the same no matter whether the angle is equal or unequal. An angle is always specified thus: 1 L $4'' \times 3'' \times \frac{3}{8}'' \times 10'\ 0''$, which reads one angle measuring 4 in. by 3 in. by $\frac{3}{8}$ in. thick by 10 ft. long. In the case of an unequal angle the longer leg is given first. The weight of an ordinary angle is never mentioned on drawings; the sizes of the legs and the thickness provide sufficient information.

A practice, fortunately rare, is to specify an equal angle of, say, $4'' \times 4'' \times \frac{3}{8}''$ as $4'' \times \frac{3}{8}''$. The saving of time thus effected is negligible, while the resulting mistakes through misinterpretation are certainly not.

The toe of the angle is shown to a larger scale in Fig. 9. From the position of the centre (obtained from Table 3, Vol. III.) of the segmental outline it is obvious that **Fig.** 10, which is the beginner's usual conception, is wrong. The root radius is larger than the toe radius, but these are constant for different thicknesses of the same angle. Further, all angles the sums of whose flanges or legs give the

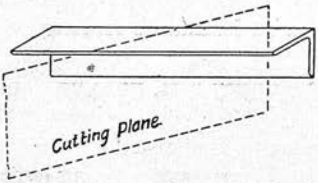


Fig. 11.

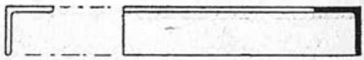


Fig. 12.

Fig. 13.

same total have similar root and toe radii; *e.g.*, for a $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ and a $4'' \times 3'' \times \frac{1}{2}''$ angle the total is 7 united inches.

To facilitate the reading of a drawing it is often advisable to indicate a break in the section. The broken surface is generally filled in with ink to emphasise the outline. The end view of an angle or any other section is not ink-filled. Fig. 11 demonstrates how the imaginary cutting plane should always be on the skew, while Figs. 12 and 13 show the resulting effects upon an angle.

Length of Angles.—Long angles are unwieldy to work with in the shops, and, moreover, if of light section, are apt to arrive there badly twisted in shipment from the mills. Wherever possible lengths should be specified upon which there is no extra to pay.

Stockyards (*i.e.*, people who buy in quantity from the mills to supply small demands) generally stock most sections up to 40 ft. in length, which forms quite a fair maximum to work.

Thickness of Angles increases by $\frac{1}{16}$ in., although many steel merchant tables only give the list of heavier sections as increasing by $\frac{1}{8}$ in. The various properties of the $\frac{1}{16}$ in. increase of thickness can be closely arrived at by interpolation.

Thick Angles.—Angles can be rolled to a greater thickness than the maximum specified in the tables, and can be readily obtained if the tonnage order is sufficiently large. The extra thickness of metal is obtained by opening out the rolls.

As Fig. 14 shows, the heel of the bar is still sharp in profile, but the toes are slightly rounded on the outside; this will be understood by reconsulting the diagrammatic elevation of the finishing rolls in Fig. 1.

Thick angles generally prove themselves troublesome in constructional steelwork, especially so if they are long and require to be spliced. The thick leg and sometimes thicker cover leave very little room for the rivets in the adjoining leg. It is impossible to use an internal cover in the case of the $3'' \times 3'' \times \frac{5}{8}''$ angle shown in the dotted outline of Fig. 15, as dimension B is too small, whereas dimension A, of the $3'' \times 3'' \times \frac{3}{8}''$ angle (hatched cross-section), permits both the use of an internal cover and the closing

Widening out rolls to roll thicker angles. Standard section hatched

FIG. 14

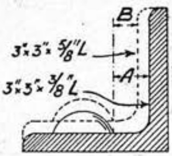
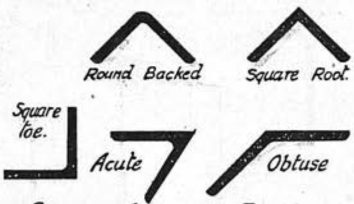


FIG. 15.



SPECIAL ANGLES. FIG. 16.

of the $\frac{3}{4}$ inch diameter rivet in its standard position. Both angles are British Standard equal angles.

Weight of Steel is taken at 489·6 lb. per cubic foot. From this an extremely useful figure is obtained for the running out of weights; viz., a bar $1'' \times 1''$ in cross-section weighs 3·4 lb. per foot run.

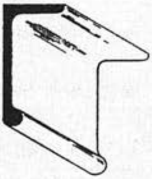
A rapid and much-used rough approximation is to take the cross-sectional area and mentally shift the decimal point one place to the right (*i.e.*, multiply by 10) and then divide by 3. The error is $3{\cdot}4 - 3{\cdot}33 = 0{\cdot}07$ lb. per square inch per foot run.

Area and Weight of Angles.—The cross-sectional area of an angle is made up of two rectangles, which in the case of Fig. 7 are $4'' \times \frac{3}{8}''$ and $3\frac{5}{8}'' \times \frac{3}{8}''$. The cross-sectional area is therefore $7\frac{5}{8}'' \times \frac{3}{8}'' =$ 2·86 sq. in., the extra metal at the root fillet balancing the loss at the toes. The weight per foot $= 2{\cdot}86 \times 3{\cdot}4 = 9{\cdot}72$ lb., which is the listed weight. Using the approximate method, the weight per foot $= 28{\cdot}6 \div 3 = 9{\cdot}54$ lb. Tables giving united inches and weights are based upon the preceding rule of rectangles. If two or more angles (and tees) have the same thickness, their weights and areas are similar, provided that the flange sums (in united inches) are the same. The following angles have the same sectional area :

$4'' \times 4'' \times \frac{3}{8}''$, $5'' \times 3'' \times \frac{3}{8}''$ and $4\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$, since the united inches are 8.

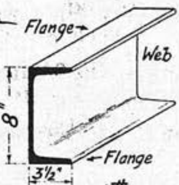
Special Angles.—The cross-sections of five special angles are illustrated in Fig. 16. These naturally are more difficult to obtain, and may cost from 10*s.* to £2 per ton over the ordinary angle basis price. An immediate consequence of this is that where a small length of a special angle is indicated an ordinary angle is planed or smithed to suit. A common feature to find on drawings is a round-backed angle serving as an internal cover at an angle splice. Instead of ordering this angle, the constructional works plane the heel away from another angle, in order that the angle cover may fit into the root of the main angle; alternatively a plate of the specified thickness may be bent to suit. See p.82 .

Again, should a drawing demand a $2\frac{3}{4}'' \times 2\frac{3}{4}'' \times \frac{3}{8}''$ angle (which is a British Standard section), and inquiry prove that it may be



BULB ANGLE.
FIG. 17

CRANE GIRDER.
FIG. 18.



8"×3½"×20¼#
FIG 19.

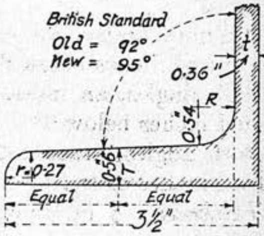


FIG. 20.

several weeks before this section is obtainable, the constructional works, instead of waiting, may replace the specified section by a $3'' \times 3'' \times \frac{3}{8}''$ angle. Permission is generally sought to thus replace a section. On the other hand, the $2\frac{3}{4}'' \times 2\frac{3}{4}'' \times \frac{3}{8}''$ size of angle may be imperative if there is not sufficient clearance to use the larger section. Provided the quantity is not excessive, the usual procedure is to plane away the toes of a $3'' \times 3'' \times \frac{3}{8}''$ angle and reduce it to a square-toed angle (Fig. 16) measuring $2\frac{3}{4}'' \times 2\frac{3}{4}'' \times \frac{3}{8}''$. The root radius of the "manufactured" angle is slightly larger than that of the angle specified, but that is of no importance whatsoever. A considerable saving in time is effected by the operation described at an expense somewhere in the region of $\frac{1}{4}d.$ to $\frac{1}{2}d.$ per lineal foot of angle.

Bulb Angles.—Due to their being used in shipbuilding, these sections can be much more readily obtained than the special angles mentioned above. Certain scantlings (*i.e.*, sizes) are at the ordinary angle basis price. For outline see Fig. 17.

The bulb helps to stiffen the outstanding leg, especially when the angle is under compression along its length. The small crane girder of Fig. 18 has the upper, or compression, flange formed of two of these bulb angles. A small depth horizontal girder, whose flanges are the bulbs, is thus formed to counteract the bending action which sometimes occurs when the craneman, instead of using a direct lift and cross travel, drags the load across the shop floor in a direction normal to the girder length.

Extras Charged on Angles and Bulb Angles.

Length.—No extras for lengths between 10 ft. and 60 ft.

Extra for over 60 ft. long.

Increasing extra for decrease in length below 10 ft., 10 ft. to 5 ft., 5 ft. to 3 ft., and under 3 ft.

United Inches.—*I.e.*, sum of the legs or flanges (8 in. in the case of Fig. 7).

Ordinary angles between 6 and 12 united inches, basis price.

Bulb angles between 9 and 12 united inches, basis price, including $9'' \times 3\frac{1}{2}''$.

Ordinary angles, an increasing extra for each 1 in. decrease in the united inches below 6.

Bulb angles, an increasing extra for each $\frac{1}{2}$ in. decrease in the united inches below 9.

Both angles, an increasing extra for each 1 in. increase in the united inches over 12.

Thickness.—$\frac{3}{8}$ in. thick and up, both types of angles, at basis price.

Increasing extra charged for every decrease of $\frac{1}{16}$ in. in the thickness below $\frac{3}{8}$ in. Some firms quote as basis price $\frac{1}{4}$ in. thick and up.

It does not necessarily follow that because a $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ angle has an extra to pay ~~that~~ it is cheaper to use the larger angle, $3'' \times 3'' \times \frac{3}{8}''$. If the smaller angle fulfils the purpose, it should be used, as it is cheaper both because of initial cost and decreased dead load of the structure. Small lengths of any one section are added together and ordered as one long bar.

CHANNELS (Symbol [)

The channel section is an extremely useful one in structural design. Channels are listed as depth by flange width by weight in pounds per foot run. When the weight comes to an odd decimal the nearest quarter-pound is usually quoted on the drawings. The 5 per cent. rolling margin is sufficient justification for this practice. Fig. 19 gives a cross-section of an $8'' \times 3\frac{1}{2}'' \times 20\frac{1}{4}$ lb. channel; as listed it weighs 20·21 lb. per foot.

One flange of a $10'' \times 3\frac{1}{2}'' \times 24\frac{1}{2}$ lb. channel (24·46 lb.) is drawn to a large scale in Fig. 20. Angles have parallel-faced legs, but the channel has the inner face splayed at an angle of 95 degrees to the web. In no case is the web thicker than the flange. The thickness of the latter is measured at a point half-way between the inner web face and the flange toe, and not midway between heel and toe. Various steel merchants' catalogues usually assign the symbols shown to the various radii and thicknesses.

A draughtsman's "trade trick" is to mark his clinograph or adjustable set square with two incisions, as in Fig. 21. By adjusting the outer edge of the swing arm, co-linear with the knife marks, the splays of joists and channels are at once obtained. This, even in the course of a year, means a considerable saving of time. Others, again, plane down old or broken celluloid set squares to the requisite bevel.

Length of Channels.—The remarks on angle bars also apply here.

Thick Channels.—By opening out the rolls a heavier scantling of channel is obtainable. The extra metal is given along the back or heel side of the web, so that the web is thickened up and is suitable for use with heavy shearing forces, where depth of headroom is a vital consideration. The flanges are therefore slightly lengthened along the outer faces, but otherwise unaltered.

Special Channels are obtainable, but are seldom used in constructional steelwork..

Extras Chargeable on Channels over Angle Basis Price.

Length.—No extra for length on lengths from 10 ft. to 60 ft.

Extra over 60 ft.

Increasing extra for decrease in length below 10 ft., 10 ft. to 5 ft., under 5 ft. to 3 ft., and under 3 ft.

Web.—Flanges 3 *in. wide or over* and web $\frac{3}{8}$ in. thick or over.

Depth of web of channel 6 in. to 12 in. at minimum price, which is about 5*s*. per ton over angle basis price.

Extra if web is from 12 in. to 15 in. deep and heavier extra over 15 in. deep.

Web.—Flanges any width and web $\frac{3}{8}$ in. thick or over.

Rising extra for each decrease of 1 in. in the web depth below 6 in.

Web Thickness.—Increasing extra for each $\frac{1}{16}$ in. decrease in web thickness below $\frac{3}{8}$ in.

JOISTS (Symbols **I** and H)

Joists are also known as beams, **I** beams, H beams, or as R.S.J.'s, the usual contraction for rolled steel joists.

This section with channels and angles form the triumvirate of sections in greatest demand. The joist is especially favoured by architects and builders to carry floors, door lintels, stair treads, etc., and is more used by them than any other steel section in ordinary brick or stone and lime construction.

R.S.J.'s are specified in a similar manner to channels, viz., depth by flange width by the weight per foot in pounds; see Fig. 23. Unlike channels, however, the listed weight is without decimals. The set of the inner face of the joist flange is 98 degrees to the web, and is constant for the various British Standard beams (Fig. 24).

Thick R.S.J.'s can be obtained by lifting the rolls, a practice which is not nearly so common as with the previous sections. The extra metal is added centrally along the web. The flanges are

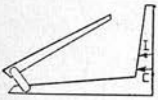
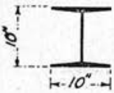
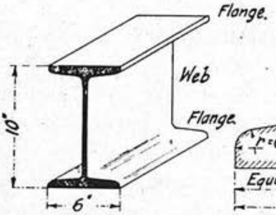


FIG. 21.



Broad flanged beam.
FIG. 22.



10"×6×40# R.S.J.
FIG. 23.

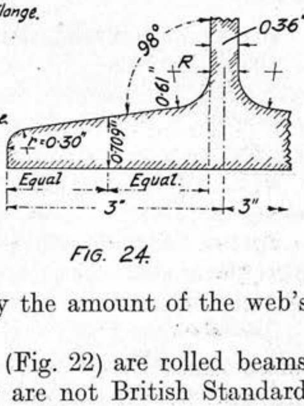


FIG. 24.

unaltered except that they are wider by the amount of the web's increase of thickness.

Special R.S.J.'s.—Broad flange beams (Fig. 22) are rolled beams with exceptionally wide flanges. They are not British Standard joists, and, although rolled abroad, can be obtained in that quality of steel which meets the requirements of the British Standard Specification. In some sections the flanges are not tapered, but are parallel, while the toes are square.

A few sections come from America, but the majority of them are of Continental origin. A broad flange beam may be listed as a $10'' \times 10''$, whereas it really measures $9\frac{27}{32}'' \times 9\frac{27}{32}''$ (*i.e.*, 9.843 in.), the decimal units being 25 cm. × 25 cm. Under certain conditions they possess slight advantages over the British Standard "B" heavy beams or pillars, and their popularity would doubtless grow were they rolled in this country.

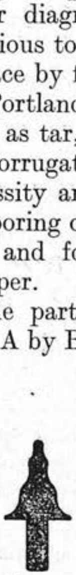
Extras Chargeable on R.S.J.'s.

Length.—Sections 10 ft. to 40 ft. long, no extra for length (some firms 10 ft. to 50 ft.). Extra (about 1*s.*) per ton for every foot or fraction of a foot over 40 ft. long.

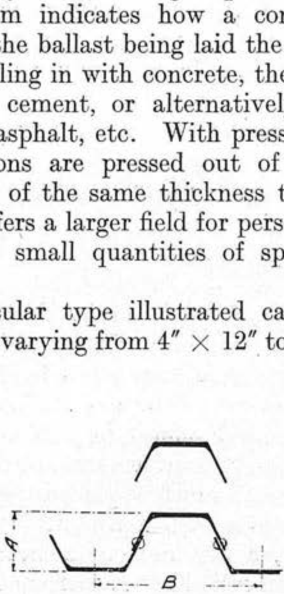
Increasing extra for decrease in length below 10 ft.

Fig. 30.—The upper diagram is a cross-section of a rolled steel trough employed in flooring, especially for railway bridges. The lower diagram indicates how a continuous length is obtained. Previous to the ballast being laid the floor is brought up to a level surface by filling in with concrete, the matrix of which is generally of Portland cement, or alternatively of a bituminous material such as tar, asphalt, etc. With pressed steel flooring the buckles or corrugations are pressed out of heated steel plates, and of necessity are of the same thickness throughout. The latter form of flooring offers a larger field for personal ideas than rolled troughing, and for small quantities of special outline is considerably cheaper.

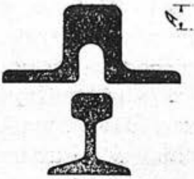
The particular type illustrated can be had in different sizes, viz., A by B, varying from 4″ × 12″ to 1′ 1″ × 2′ 8″; the respective



SASH & ASTRAGAL BARS. FIG 29.



TROUGH FLOORING. FIG. 30.



RAILS. FIG. 31.

COPE BAR, ETC. FIG. 32.

weights per square foot of covered area are 13·4 lb. and 28·8 lb., and the section moduli are 4·9 in. and 72·7 in. cubed.

Fig. 31.—In addition to the bridge rail and flat bottom rail which are illustrated, there are the tram rail, the heavy crane rail, and the bull-headed rail. Nearly every railway has its own particular type of the latter form of rail. Rail sections when worn prove extremely useful as reinforcement for grillage work in foundations.

Fig. 32.—Cope bars and hollow half-rounds are useful in making hand railings. Cope bar makes an ideal attachment for corrugated sheeting. See plate II. Vol II. Dimensions A × B can be had from $\frac{1}{4}'' \times \frac{3}{4}''$ up to $1\frac{1}{2}'' \times 7''$.

Round Bars.—In the basements of shops and warehouses ~~several London shop~~s, where space is extremely valuable, steel columns are made of large diameter solid steel round bars.

The caps and bases are also of solid steel and have holes bored out of them slightly less in diameter than the finished diameter of the turned ends of the shaft.

Both cap and base are then heated and fitted on to the shaft, *Depth.*—R.S.J.'s, 5″ × 3″ up to 14″ × 6″, basis price.

Notwithstanding the above, the wider-flanged joists, 10″ × 8″ and 9″ × 7″, carry an extra.

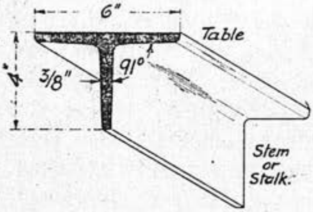
Increasing extra for each increase or decrease outwith the specified basis depths.

TEES (Symbol T)

Tee bars are specified by giving the table dimension by the stem dimension by the thickness, as in Fig. 25.

The most useful sections are the smaller sizes of tees which are used for astragals in ordinary putty glazing for side and roof lights. These sections are $1\frac{1}{2}'' \times 1\frac{1}{2}'' \times \frac{1}{4}''$, $1\frac{3}{4}'' \times 1\frac{3}{4}'' \times \frac{1}{4}''$ and $1\frac{1}{2}'' \times 2'' \times \frac{1}{4}''$.

At one time tees were greatly used for the main rafters and struts of roof trusses, but their employment entailed eccentric riveted

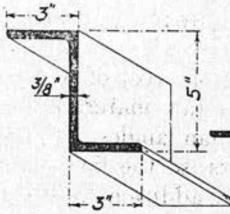


6"×4"×3/8" TEE BAR.

FIG. 25.

BULB TEE.

FIG. 26.



ZED BAR.

FIG 27

FIG. 28.

gusset connections, which could only be eliminated by cutting away either the stem or the table. In the smaller sections little clearance was obtainable for riveting. Modern practice has forsaken T-bar rafters for two angles back to back.

Weight of T-bars.—Angle bars and tee bars have the same weight per foot approximately if their united inches and thickness are the same, viz., the $6'' \times 4'' \times \frac{3}{8}''$ tee of Fig. 25 has the same weight per foot as a $6'' \times 4'' \times \frac{3}{8}''$ or a $5'' \times 5'' \times \frac{3}{8}''$ angle, the united inches being 10.

Special T-bars are rolled for several patent glazing firms, the rolls having been cut to their requirements. These special tees are sometimes covered with a metallic covering of lead or tinned lead.

Bulb Tees (Fig. 26) are rarely found in shop or bridge construction.

Extras Chargeable on all Tee Bars over Angle Basis Price.

Length.—No extra on lengths between 10 ft. and 60 ft.

Extra over 60 ft.

Increasing extra for decrease in length below 10 ft., 10 ft. to 5 ft., under 5 ft. to 3 ft. and under 3 ft.

which they tightly grip on cooling and shrinking. If not properly aligned, the load may act eccentrically in the column, causing a bending stress as well as a direct thrust. Only in exceptional circumstances is such an uneconomical column used.

Other Extras on Sections.—The more work which is expended on the rolled sections at the mills over and above the usual mill finish the greater is the extra tonnage rate.

Cold-straightening is included in the basis price of rolled steel joists, but is an extra on all other sections. Constructional works always cold-straighten sections which may arrive slightly twisted, otherwise holes would not be in register.

Oiling, Painting and Annealing if done at the mills must be paid for.

Tests and Tensile Strength etc.—Any departure from the standard routine and specification, which involves special attention at the steel company's works, produces a corresponding extra.

Cutting bars to length is done when the bars are hot to a margin of 1 in. under or over the specified lengths. Alternatively the manufacturer may be given a 2 in. cutting margin over the specified length, but nothing under this length. (See also the B.S.S. for Girder Bridges, Materials and Workmanship.)

Exact Length.—Cutting by hot saw prevents accuracy of marking off lengths, so that if exact lengths are ordered the bars are cold-sawn. Manufacturers define the word "exact" as meaning to within the nearest $\frac{1}{8}$ in. over or under; others, again, quote $\frac{1}{16}$ in. over or under. Extras are charged by the mills for this. The constructional works do this exact cutting to length themselves, and in consequence seldom order anything from the mills other than with the usual mill finish.

Common Sections.—Although an angle or any section is given by the British Standards Committee in their list, it does not necessarily follow that this section can be easily obtained. Practice has found several particular sections extremely useful, with the result that a demand is created for these sections. The steel-makers concentrate on these sections to the detriment of others on their list. Some sections may thus be rolled once a week, or once a month, or perhaps only once a year.

The changing and setting up of the rolls, which may cost, say, £50, will only be done when there is a sufficiently large order on hand to justify the expense. Again, it might even be worth spending several hundreds of pounds in getting special heavy rolls made and turned provided that the rolling is sufficiently large. The larger the rolling the less is the cost of the rolls per ton of output. To balance this cost, there is probably a saving in drilling, riveting, planing, etc., of a built-up section. In the case of the strengthening

United Inches, *i.e.*, sum of table and stem, which for Fig. 25 is 10 in.

Ordinary tees between 6 and 12 united inches at the lowest price per ton (approximately £1 over the angle basis price per ton).

Increasing extra per ton for each inch or part outwith the limit of united inches given above.

Thickness.—$\frac{3}{8}$ in. thick (some firms $\frac{5}{16}$ in.) and up, without extra for thickness.

Every $\frac{1}{16}$ in. decrease below this thickness adds an extra to the tonnage price.

ZED BARS (Symbol Z)

Zed bars are specified as depth by flange by thickness; Fig. 27 therefore shows a $5'' \times 3'' \times \frac{3}{8}''$ zed bar. Their use in this country is practically "tabooed" by several constructional firms, whereas in the United States the reverse is the case. The former firms would apparently prefer to spend money on drilling and riveting angles, after the manner of Fig. 28, rather than use rolled zed bars. Against the cost of workmanship must be placed the saving in the price of raw material, since zeds cost approximately 10*s*. per ton more than angles. Further, angles are commoner sections than zed bars, so the final built-up zed shape is more quickly obtained, with the additional advantage of a great range of section depth and section modulus. The zed type of section, rolled or built, is mostly used for purlins.

Extras Chargeable on Zed Bars over Angle Basis Price.

Length.—No extra for length on lengths between 10 ft. and 40 ft.

Extra for every foot or part over 40 ft.

Increasing extra for each decrease in length below 10 ft., viz., under 10 ft. to 5 ft., under 5 ft. to 3 ft. and under 3 ft.

United Inches, *i.e.*, sum of flanges plus depth.

Cheapest rate 10 in. to 14 in., about 10*s*. per ton over angle basis price.

Rising extra for each increase of 1 in. or part over 14 united inches.

Rising extra for each decrease of $\frac{1}{2}$ in. or part under 10 united inches.

Web Thickness.—Increasing extra for each decrease of $\frac{1}{16}$ in. or part below $\frac{3}{8}$ in. thick.

MISCELLANEOUS ROLLINGS

Fig. 29.—Sash bars and astragals have many varied forms, as will be found from the catalogues of the firms who specialise in this work.

Fig. 30.—The upper diagram is a cross-section of a rolled steel trough employed in flooring, especially for railway bridges. The lower diagram indicates how a continuous length is obtained. Previous to the ballast being laid the floor is brought up to a level surface by filling in with concrete, the matrix of which is generally of Portland cement, or alternatively of a bituminous material such as tar, asphalt, etc. With pressed steel flooring the buckles or corrugations are pressed out of heated steel plates, and of necessity are of the same thickness throughout. The latter form of flooring offers a larger field for personal ideas than rolled troughing, and for small quantities of special outline is considerably cheaper.

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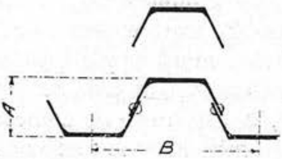
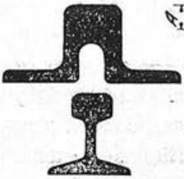
SASH & ASTRAGAL BARS. FIG 29.



TROUGH FLOORING. FIG. 30.



RAILS. FIG. 31.

COPE BAR, ETC. FIG. 32.

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Fig. 31.—In addition to the bridge rail and flat bottom rail which are illustrated, there are the tram rail, the heavy crane rail, and the bull-headed rail. Nearly every railway has its own particular type of the latter form of rail. Rail sections when worn prove extremely useful as reinforcement for grillage work in foundations.

Fig. 32.—Cope bars and hollow half-rounds are useful in making hand railings. Cope bar makes an ideal attachment for corrugated sheeting. See plate II. Vol II. Dimensions A × B can be had from $\frac{1}{4}'' \times \frac{3}{4}''$ up to $1\frac{1}{2}'' \times 7''$.

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The caps and bases are also of solid steel and have holes bored out of them slightly less in diameter than the finished diameter of the turned ends of the shaft.

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Other Extras on Sections.—The more work which is expended on the rolled sections at the mills over and above the usual mill finish the greater is the extra tonnage rate.

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The changing and setting up of the rolls, which may cost, say, £50, will only be done when there is a sufficiently large order on hand to justify the expense. Again, it might even be worth spending several hundreds of pounds in getting special heavy rolls made and turned provided that the rolling is sufficiently large. The larger the rolling the less is the cost of the rolls per ton of output. To balance this cost, there is probably a saving in drilling, riveting, planing, etc., of a built-up section. In the case of the strengthening of the floor of the Forth Bridge (*Minutes of the Proceedings Inst. C.E.*, Vol. CCXV., 1922–23, by W. A. Fraser, M.Inst.C.E.) a rail trough composed of three rolled steel channels was adopted. These channels, which are 17 in. deep, had to be specially rolled, but the resulting trough is an extremely rigid one, and was obtained with the minimum amount of labour. Again, patent glazing companies have rolls especially cut to roll their astragal bars, not a costly undertaking with light sections. The heavier the section the greater the cost of cutting the rolls and changing them in the mills.

Choice of Sections.—Naturally the previous lists of extras make an imposing array, which somewhat dismays the beginner. The simplest rule for the embryo designer is not to think too much concerning extras, but rather choose sections which can be readily obtained. As a help in this direction the sections given in the tables of Vol. III. have been arranged so that no difficulty should be experienced in picking out the commoner sections. With the above rule may be coupled the remark that, broadly speaking, angles, joists and channels generally prove themselves the cheapest and most efficient of all sections.

REFERENCES

Steel Rolling Mills.—*Vide* the *Journals of the Iron and Steel Institute*, which give descriptions of various types of mills, gearing, power used, practice and methods, etc. The index volumes of the Institute give a comprehensive list of other publications dealing with this subject.

Standard Sections.—*Dimensions and Properties of British Standard Rolled Steel Sections for Structural Purposes*, issued by the British Engineering Standards Association of 28, Victoria Street, London. This book is the source of the properties list of standard sections given by the various steel companies. The books issued by these companies include many sections rolled by them which are not standard, but nevertheless useful.

Section Books.—The following books give a list of the properties of sections in addition to much other useful information; in fact, in one or two cases these books approach the text-book standard, so copious are the notes and data. They can usually be obtained free on payment of postage, but much depends upon the position in business occupied by the applicant. This list is by no means a complete one :—

Consett Iron Co., Ltd., Consett Co., Durham (chequered plating, etc.).
David Colville & Sons, Ltd., Motherwell, Scotland.
Dorman, Long & Co., Ltd., Middlesbrough, England. (on sale).
Glengarnock Iron and Steel Co., Ltd., Ayrshire, Scotland.
Lanarkshire Steel Co., Ltd., Motherwell, Scotland.

Redpath, Brown & Co., Ltd., Edinburgh, Glasgow, London and Manchester.

Steel Co. of Scotland, Ltd., Royal Exchange Square, Glasgow.

Continental and British sections, see *Structural Steel*, the handbook of Messrs. R. A. Skelton & Co., Moorgate Station Chambers, London, E.C. (on sale).

American sections, see the *Cambria* section book of the Cambria Steel Co., Philadalphia, U.S.A. (on sale).

CHAPTER II

THE DRAWING OFFICE

Tenders and Estimates.—In point of fact, the original designer is the client, whether or not he possesses any knowledge of stresses and strains. The client desires a footbridge or some other structure erected to fulfil a number of conditions. It has to be a certain length, a certain width, a certain height, and so on; money may or may not be spent profusely upon it, and its useful life may be fixed or indefinite. The client then turns to an architect, a consulting engineer, or sometimes directly to the constructional works, whosoever the client may deem best fitted for the particular job on hand.

The job is now considered in more detail: loads are calculated, resulting stresses found, and the sections finally arrived at. A design drawing is now made, usually to a small scale ($\frac{1}{4}$ in. to $\frac{3}{8}$ in. to 1 ft., depending upon the size of the structure), but all important connections and details are drawn separately to a larger scale (from $\frac{3}{4}$ in. to $1\frac{1}{2}$ in. to 1 ft.), so as to ensure that the preliminary design is feasible.

Very often at this stage consulting engineers and architects have the drawings traced in ink upon tracing cloth or paper. Blue prints are made and sent to the steel constructional works, together with a specification detailing the quality of the materials and workmanship, while the covering letter asks for a price to be tendered before a certain date.

Specifications.—The best method of specifying is to quote "that the work shall comply in all respects with the requirements of the British Standard Specification for Girder Bridges No. 153, Parts 1, 2, 3, 4 and 5 of 1922–23," and to amplify it by an accompanying enclosure. The latter should state which of the alternative courses or methods laid down in the standard specification shall be followed. This standard specification is eminently just to manufacturer and purchaser alike.

Special clauses are sometimes inserted which, if fully carried out, would virtually call for as much meticulous care and refinement in workmanship as is required in the manufacture of some delicate piece of mechanism. This can, of course, be done, but at an enhanced cost; the work can be carried out just as well and efficiently without such unnecessary refinement. What is wanted is an honest, straightforward job, such as is called for in the standard specification.

Time Clauses binding the delivery or completion down to a certain day are but natural; but when the time allowed is unreasonably short and the penalty high the tendered prices are increased to cover the risk. Self-preservation is Nature's first law. If the time set apart for the job is short, then the manufacturer must obtain his material at once, and to do so offers higher prices, buying from stock, from mills, anywhere, in order to obtain speedy delivery. It is by no means uncommon for the constructional works to be asked to give (*a*) a tender, based upon incomplete drawings, within a week, and (*b*) to manufacture and erect a job in less time than that occupied by the design. The client presses for delivery, and the time lost in the design office has to be recovered elsewhere.

Incomplete Drawings.—No words can fully convey the trouble entailed by this system of tendering on incomplete drawings. The unfortunate person who has to "take out" the weight, as a step towards ascertaining the cost of the structure, must allow for pieces and connections which are not shown; in other words, he has to guess (scientifically, no doubt, but nevertheless guess), and should he guess low, the firm probably secures the job. Generally, however, the uncertainty tends to keep the cost high.

On the other hand, the firm may knowingly cut their price, and this can only be done at the expense of weight and strength. Advantage is taken of the incomplete drawings to use slender scantlings, and the greater the temerity of the firm concerned the lighter the sections. That this is by no means exaggerated the following items, taken from competing tenders for the same building, show: roof trusses, span about 43 ft. at 11 ft. centres; purlins approximately at 6 ft. centres; length of building, 200 ft.

Firm A:

6 purlins @ $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}'' \times 200'$ @ 8·45 lb./ft. =		10,140 lb.
4 purlins @ $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{5}{16}'' \times 200'$ @ 7·11 lb./ft. =		5,688 lb.
	Total	15,828 lb.

Firm B:

10 purlins @ $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}'' \times 200'$ @ 4·04 lb./ft. = 8,080 lb.

The purlins of firm B must presumably act as suspended cables, and not as beams!

Designs direct from Steelworks.—It occasionally happens that the Consulting Engineer or Surveyor, etc., having only a slight knowledge of structures, sends inquiries direct to several constructional firms. There is enclosed a single line diagram of the proposed bridge or structure, together with overall dimensions and the loads which the structure has to carry. He asks for drawings indicating the contractor's proposals, together with the price inclusive of erection. This method gives a still wider range for thinning down. Firms are competing on the vastly different levels set by their anxiety to get work and by the scientific knowledge of their staffs. One firm employs highly specialised men for designing, whereas another employs perhaps a single man, who has to turn his hand to designing, draughting, ordering of material, etc. The costs of these staffs are entirely different, and make themselves felt in the prices. It is a fact that the unskilled man nearly always designs light; he cannot foresee all the forces or loads which may act, and generally thinks of standard cases of loading.

The question now naturally arises in the reader's mind, "But if the light structure stands up, why put up a heavier and therefore a more costly one?" It is true that there is much to be said for this, but the reader would probably prefer to cross a bridge which is absolutely safe than one which is just on the margin of safety. A lightly designed structure is seldom rigid; and rigidity in structures is a valuable asset, but involves, as a rule, increased weight. A structure may sway and deflect alarmingly and yet stand up. As a case in point, a Continentally manufactured tower crane was erected in this country which on testing carried its test load safely, but the deflections were so great that the working loads had to be considerably reduced from those originally intended. The factors of safety and good luck also help these "cut" structures, but in the long run they are not economical, and parts have to be renewed much sooner than would otherwise have been the case. After all, it is only common-sense to say that by dealing with a reliable firm the chances are in the client's favour.

The Engineer having received the various tenders and drawings, accepts, as a rule, the lowest or one of the lowest offers, but this does not necessarily mean the cheapest structure, since if it has been too lightly designed the upkeep will swallow up any saving in initial cost. This method of operation lends itself to some other objectionable practices, and a case is given in illustration. X. asked for and obtained prices and designs from various competing firms, and among these was a good set of dimensioned drawings from a firm of repute. This firm lost the contract, but were amazed to see their own design being erected. Inquiry proved that X., liking the design of the firm, had arranged with another firm to undertake it, with minor alterations, at a much lower figure.

Complete Drawings.—The Consulting Engineer should make complete drawings, and these, together with a good specification, form the only satisfactory basis of tendering. He has had time to weigh up the pros and the cons, and should have conveyed the job to paper exactly as he desires it to be before prices are asked. It is only fair to say that the latter is the practice of the majority of engineers, especially those belonging to the recognised institutions of engineers, but unfortunately any person, no matter what his lack of qualifications is, can call himself a "Consulting Engineer," and therein lies much of the trouble. The competing steel contractors, having been furnished with the drawings and specification, are all at "scratch" in the race; their yard equipment, their contracts with the rolling mills or with the stockyards for material, their erection schemes and transport of the work to the site, all furnish legitimate fields wherein money may be saved without danger to the client, because prices only become comparable when quoted for the same set of drawings, specification, and schedule of quantities.

The more detailed the drawings submitted for prices the less are the chances of their being misread wilfully or unintentionally, and the less is the work entailed in estimating from them. A point which seems to be overlooked is, that if for one job eleven firms are asked to send in designs and tenders, then ten of them must spend money needlessly. It costs each of the competing firms, say, £50 to make out their estimate (on some occasions it has reached thousands for a single estimate and competitive design), making a useless and non-productive expenditure of £500. Who pays this? The only reply is, "The client himself," because he in turn is paying an enhanced price for some previous client's estimate. It follows that if every client who desired a price submitted a careful set of drawings, specifications and schedule of quantities, the cost of tendering would be less, and prices would tend to fall. Under the latter system only one office, that of the Engineer, does the work, and not eleven independent ones.

Although these practices are not common, the mere fact of their existence calls for a warning to be given to the student, and it is because no text-book draws attention to them that the writer has felt himself constrained to write rather fully upon them, knowing that he has the approval of many engineers in so doing.

Detailed Drawings are based upon the small scale design drawings, and should have been included amongst those sent out when asking for prices. These drawings show the different pieces of the structure to a large scale. Take, as an example, the case of a workshop. The design drawing of $\frac{1}{4}$ in. to 1 ft. scale would show the cross-section, outside elevation and plan, and any additional views necessary. The roof truss on this drawing would have the sections written against each bar, together with gusset plate thicknesses. The rivets would not be shown as such, but would be indicated at the end of each bar as 3 r., 2 r., etc., the diameter of the rivet hole being specified in a prominent footnote.

The draughtsman, under the superintendence of the designer, redraws the truss on one sheet (or sometimes half the truss when it is symmetrical).

The scale of this detail drawing is usually $\frac{3}{4}$ in. or 1 in. to 1 ft., while special details may be enlarged to $1\frac{1}{2}$ in. to 1 ft. This sheet would then give the main or leading dimensions of the truss, such as height or rise, and span, etc. This fixes the outline of the truss, and the smaller bars, forming the web or interior portion of the truss, then automatically fall into position. Because these subsidiary bars are, with regard to length, functions of the leading dimensions, some drawings do not quote their length. The exact fixing of length is thereupon left to the template-maker to measure from his full-size lay-out.

The lengths of these bars can be scaled from the drawings fairly accurately, and should the draughtsman want to "cover" himself, he can always add after the dimension the contraction "abt." for "about." To a draughtsman familiar with "logs" there is no great difficulty in calculating their lengths. The point is that by noting the lengths of the secondary bars on the drawing the draughtsman facilitates the ordering and sorting out of material. The system of wood template making has encouraged this habit of not stating the lengths of subsidiary bars.

The drawing shows every hole and rivet, and fixes their position by giving dimensions to some important point in the structure, such as a panel point, etc. Holes which are to go open to the site are ink-filled on the drawing to differentiate them from holes which are riveted up in the shop.

The scantling and size of every section is mentioned alongside the bar concerned, and all printing and dimension lines are kept clear of the actual drawing itself. It is better to err on the full side with information, but repeating obvious information results in a congested drawing which is difficult to read.

Usually this drawing is made on a cheap cartridge paper or a thin detail paper in pencil line. It is next forwarded to the tracing office, where girls trace the drawing on tracing cloth. In some offices the drawing is made straightway upon the tracing cloth, the dull side of the cloth being used, as it takes a pencil line. The lines about which there is no dubiety are inked in at once, the pencil work keeping pace until the whole tracing is completed.

Ordering Material.—The foregoing detailed drawings may be made previous to entry of the job to the constructional works or after entry. In any case a job number is assigned to it, and all subsequent time and material is charged against this job number.

Material can now be ordered either from the mills or from the steel merchants' stockyards with every degree of accuracy. The sections are grouped under their respective scantlings, lengths of small pieces summed and ordered as long bars.

Joist scrap, *i.e.*, cuttings and odd lengths, has little useful value in constructional works, and is sent back at scrap value to the smelters. Channel scrap is perhaps slightly more useful, but in any case these bars should be ordered from the mills as near the exact length as is possible. It is better to try to arrange the order list so that there is a large number of bars of the same length instead of every bar being of a different length, and when ordering from the mills the 2 in. cutting margin should be remembered.

Angles may be accepted, say from a steel merchant's stockyard, from several inches to several feet longer than is required for the job on hand. This depends greatly upon the section. The extra lengths are put into the stock of the constructional works and are drawn upon to make cleats, small stiffeners or short bar members of roof trusses. The important point is that odd lengths of angles have a use in the constructional works, and are not merely scrap steel.

Plates and flats give valuable scrap, their cuttings and trimmings being used for small gusset plates and washers. Small plates are cut out of long plates, and when ordered are so arranged that the small pieces fit into each other neatly, without much intervening waste space. Care is taken that this entails no re-entrant cuts. If plates are to be planed, the $\frac{1}{8}$ in. or $\frac{3}{16}$ in. allowance for planing each edge should be added to the ordered width.

The order sheets are typed through carbon papers or written by hand so that several copies are obtained. An extra column is generally given on these sheets, wherein are briefly stated the sizes into which the ordered lengths are to be subdivided on their arrival at the yard, and also the position they occupy in the structure. Obviously this is a valuable help to the yard.

In addition to the copies required by the order clerk and the drawing office for reference, copies of the material list, together with blue prints of the working drawings, are sent to the template loft and the yard.

REFERENCES

Coleman and Flood. *Civil Engineering Specifications and Quantities* (Longmans, Green & Co.).

British Engineering Standards Association. *Drawing Office Practice* (No. 308, 1927), price 2*s*.

CHAPTER III

THE TEMPLATE LOFT AND THE WORKS

THE TEMPLATE LOFT

Template-making is the next stage in the fabrication of a structure.

The roof truss, which has been already mentioned, is now drawn full size upon the template (or templet) floor from a base and a centre line. Long lengths are measured with a steel tape and are struck with a chalked cord. The actual length of every bar is then obtained from this drawing.

The templates may be either (1) the bars which are to be used in the structure or (2) made from some light and easy easily worked material.

Method 1 calls for a highly skilled plater, who does all his own measuring and setting out of the work. The templates are the actual sections after they have been straightened. One bar is therefore finished, and other bars similar to it are marked from it. The plater is thus always in touch with his job after it leaves the drawing office, and becomes more conversant with it as it approaches completion. Further, since it is the bars of the structure which are used for templates, any irregularity due to sectional growth is allowed for at once.

This practice rules in America, but has very few followers in Great Britain.

Method 2 employs wooden strips which are made to the full size lengths obtained direct from the floor. Holes are drilled in them where holes have to be drilled in the finished bar. After completion the wooden templates are despatched to the yard. Instructions are written on them in heavy pencil or paint to direct the "marking off" of the actual bars. This is the practice in this country. It has one drawback in that it reacts disadvantageously upon the drawing office, which is inclined to "leave it to the loft."

It is argued against this system that, in addition to the plater, the template-maker has to "worry out the drawings" also, to which the reply is that wood templates considerably simplify matters for the plater and save raw material from being wasted.

Briefly, then, method 1 requires more painstaking work from the D.O. than is common in this country; method 2 employs the extra man, the template-maker, with a saving in time of the D.O. It is a moot point as to which system is the better, but if practice counts for anything, it may be taken that the second method, probably due to the quality of labour obtainable, is cheaper; otherwise it would not be perpetuated.

The Templates are generally strips of wood $\frac{3}{8}$ in. to $\frac{1}{2}$ in. thick and of varying width up to 6 in. or even wider. Several strips may be fastened together by straps to give extra width or a skeleton frame when necessary. It will be understood that these strips represent the working surfaces of the bars. In the case of an angle one strip may represent one or both legs, since it functions as a transfer. The wood for good templates is pine; for cheap templates, white deal.

Thin gauge iron or steel makes excellent templates for accurate work. They are indestructible, and are affected by temperature in exactly the same manner as the parent members for which they are intended. Wood, on the other hand, is always seasoning, and climatic conditions call for care if accurate workmanship is to be maintained. Three-ply wood is good, and is naturally more trustworthy than single ply. Cardboard or millboard is also used, while brown paper has not been despised.

The loft floor is long and wide, with the template-makers' benches set round the wall. The cost of ground and the necessity for good light has caused the floors of most template shops to be at least one flight up. This has one advantage, as it keeps traffic off the floor, which is sometimes specially blackened once a year.

The remarks made about congested drawings are equally applicable to templates. The template-maker, to save wood and time, often makes a template strip do duty for an ordinary repeat part of the structure and also for special bars. The special bar may be an ordinary bar with additional holes and connections. Again, the bars may be so many left-handed and so many right-handed. It follows, therefore, that if the template is made to do duty for the lot it has a large list of instructions written on it, with the corresponding holes encircled in various colours to indicate which of them shall be taken. Some templates are like jig-saw puzzles, and, instead of saving time, lose it, incurring, as they do, journeys to the loft for the solution. Old templates are used over again by sawing down and plugging the holes. Templates, unlike patterns, are not commonly stored against future use, since structures are not standardised like mechanical parts.

THE WORKS

Straightening of Material.—While the work on the wood templates is nearing completion the raw material is being delivered from the mills. Unequal rates of cooling at the mills and transportation have caused slight bends in bars and plates, and these must be cold-straightened.

Cold-rolling flattens the plates. The machine used for this purpose has a double set of horizontal rollers, the upper tier of rolls entering the space between the lower ones, just as one would stack pipes. The rolls are capable of adjustment, and plates can be made flat or curved for steel chimneys, tanks, etc.

Cold-straightening may be accomplished by hand-hammering in the case of light sections. The bar is placed upon two wedge-shaped supports with the convex face up, the bend being at mid-span. By striking the section at the bend the result is an obvious straightening action. Machines for straightening (or bending) are built on this principle, and are therefore glorified jim-crows which are used in tramway or railway work for rails.

In the case of machines the hammer is replaced by a ram driven by hydraulic or other power, and the action is similar to that of hand-hammering. From the formula of $\frac{M}{I} = \frac{E}{R} = \frac{f}{y}$ the student can easily arrive at the value of the extreme fibre stress set up by this mechanical bending or straightening. That this straightening, by bending, causes stresses well in excess of the elastic limit, is not generally realised. It is quite possible that a tension angle in a structure may already be carrying a high tensile stress at one portion of its length previous to its being built into position. If the section which has been mechanically straightened be edge-planed, it always exhibits a tendency to return to the original bend. As with a punched hole, the only method of wiping out the internal stress is to anneal the section, a process which is unthinkable.

Astragal tee bars are placed in special end grips and are pulled out by hydraulic power, as in a tensile testing machine. The writer on one occasion, using an ordinary steel tape, worked out the stress for a $1\frac{1}{2}'' \times 2'' \times \frac{1}{4}''$ tee astragal so straightened, and arrived at the figure of 26 tons per square inch. There was a slight slip in the grips, but the fact nevertheless remains that the bar was overstretched and therefore overstressed.

Cutting Machines.—The bars are subdivided cold either by the revolving circular saw, by the cropping machine, by the shearing machine, or by the guillotine.

Circular Saw.—The heavier sections are usually allotted to this machine. Much can be done with the circular blade, as this can be set so as to cut a predetermined distance in and then return, or it can cut straight through. A $24'' \times 7\frac{1}{2}'' \times 90$ lb. R.S.J. can be cut through in fifteen to twenty minutes.

With the new mild steel disc saw an exceptionally high peripheral speed of 30,000 ft. per minute is obtained when running at 15,000 revolutions per minute. The blade can be raised and lowered at will, while the vertical plane of the blade may be inclined to the section at any desired angle and there locked. The sections can remain, therefore, always in the same position on the bed instead of being swept out into the shop, and so occupying valuable shop space when a bevel cut is required. The heaviest steel joist rolled, of 90 lb. per foot, can be cut through in about a minute and a half.

Shearing Machines have a heavy blade, about a foot long, inclined to the horizontal at an angle of 10 degrees or so. The cut is a matter of a few inches, while the number of strokes runs about twenty per minute. The plates get distorted, especially if long, due to the inclination of the descending blade when shearing its way through the plate. The resulting edge is ragged and uneven, and cuts the hands of the workers if they are not protected by rectangular pieces of leather with a slit through which the wrist passes.

The Guillotine is a larger machine than the shears, with a blade from 3 ft. to 10 ft. long and which has only a slight inclination to the horizontal. The resulting cut, if shorter than the blade, is straight and of good surface. While the plate is being adjusted in the machine the blade and the holding down beam remain stationary in their highest positions. On depressing the treadle or hand lever these descend, the beam leading, and on the cut being made return once more to their original positions. The weight of these machines runs from 4 to 30 tons.

The shearing action adversely affects the tenacity of the immediately adjoining fibres of the steel, but only for a short distance in. The punching of a hole has the same injurious effect. When sheared edges are used the rivet holes are kept further in from the edge than in the case of a rolled or planed edge. The extra allowance varies from $\frac{1}{8}$ in. to $\frac{1}{4}$ in.; see B.S.S. (4/27). A slight fissure or crack often predetermines the line of failure. With a sheared edge under tension there is always the possibility of one of the small cracks opening out and the plate tearing; under compression the cracks tend to close. The B.S.S. (2/3) permits a sheared edge on the top or compression edge of the web plate, but asks for a planed edge on the bottom or tensile edge.

Regarding Appearance.—A flange plate with sheared edges, especially when painted, is indistinguishable from a planed edge when viewed a few feet away.

Cropping Machines are, in effect, special shearing machines, with suitable openings for the various types of sections. For examp the angle cropper has a right-angled V-notch, which supports t legs against crushing when the blade is cutting through. Go clean-cut ends are obtainable under ordinary working conditio In the newer type of machine all manner of bevels, or splays, can given to the cut end of an angle or tee. This is in contradistincti to the older type, still in use, which can only give a straight cut right angles to the bar's length.

Highly splayed ends should not be used for the following reaso The position of the rivet is definitely fixed by the rule that its cent shall never be nearer the edge than $1\frac{1}{2}$ d., where d. = diameter hole. In a long splay the end of the bar is far removed from t rivet head, which tightly clenches the adjoining metal parts togeth and causes the end of the bar to bend up and leave the gusset plat This gap encourages rust action, since a painter's brush cannot obta entry. Splays are necessary in order to give clearance to some oth adjacent bar (see the drawing of roof truss details), otherwise standard right angle, straight through cut is preferable.

Marking Off.—With the bars, etc., cut to length the marking o is greatly simplified. The section, with the template clamped to i upper surface, is placed on a raised platform or bench. The hol in the wooden template guide the marking tool—a centre punc or a tube of the requisite diameter dipped in some cheap white pai or whitewash—~~and~~ so registering the position of the holes on t steel. The indent made by the centre punch is difficult to see, s that often both the punch mark and the vivid circle of colour a used conjointly.

The Punching Machine has an action almost similar to the han punch used for holing paper for filing purposes. The punche circular bite is pushed through; the punch then ascends, and th bar is pushed along until the next mark comes under the no descending punch, and so on. Specifications generally ordain tha material of $\frac{5}{8}$ in. or $\frac{3}{4}$ in. thickness and over shall be drilled.

The punch destroys the tenacity of the annular ring of metal jus surrounding the hole. Annealing restores the damaged metal to it pristine strength, but the cost of such a process is prohibitive an out of all proportion to the amount of damaged metal. Tw methods therefore present themselves. First, to punch a hole $\frac{1}{8}$ in less in diameter than the finished hole and then reamer or clea out the hole to the proper diameter. The damaged metal i removed by this process. Secondly, to leave the damaged metal i position, but when calculating the net area of the section deduc from the gross area of the bar an imaginary hole the diameter o which is $\frac{1}{8}$ in. larger than the actual diameter of the finishe punched hole. The latter method, which neglects the damaged metal still in place, rules in America and in some firms in this country.

The hole made by the punch is inclined to be slightly tapered in its length; further, since each plate must be punched separately and assembled, holes which take the same rivet cannot be in such accurate register as holes drilled from the solid with assembled plates in position.

Fig. 33 shows the assembled plates with punched holes which are not in exact register. (The holes have the taper exaggerated in the sketch.) The succeeding figure illustrates the holes after having been cleaned out or reamered for alignment, while Fig. 35 indicates how the finished rivet shank has a protuberance on it. The material for this excrescence can only be obtained from that metal which

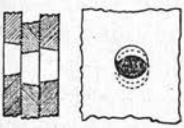


FIG 33.

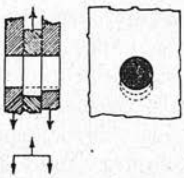


FIG 34.

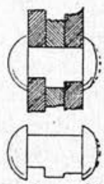


FIG 35.

should have gone towards forming the final rivet head, which must perforce now be undersized.

Drilling.—The material is removed by a high speed twist drill which leaves the metal surrounding the hole undamaged. By assembling the angles and plates which form the flange of a plate girder the drilling of each rivet hole, from the solid and through the several added thicknesses, can be carried out at one operation. Where plates, etc., can be so bundled together, marking off is considerably reduced, and, above all, the holes are in exact register, especially if sufficient tacking holes and service bolts have been used. (Service bolts are rough bolts used and reused for temporary fastenings, and are more trustworthy than clamps. The bolts are spread here and there over the job, and are taken out and replaced, etc., when too near the drill.) The plates and angles of flanges are then dismantled and all the burrs and turnings, which have been forced in between the plates, removed (B.S.S. (2/4)) preparatory to riveting.

Where gusset plates are identical these can be bundled and the holes drilled through the set at one operation. Where these plates are $\frac{3}{8}$ in. thick and under, it is actually cheaper to have the holes drilled instead of punching them ; this statement takes into account upkeep, initial cost, labour and marking off, etc. Where plates cannot be so bundled the punch is the cheaper instrument ; on the other hand, the punched holes when assembled often require the extra process of reamering to bring them into alignment.

Where several radial drills are mounted in line they form a battery of drills. The radial arm swings round the pillar as centre, while the drill itself can travel horizontally along the cantilever arm. The drill can be set to any point and then locked preparatory to drilling. This drill does all the straightforward girder drilling. In addition there is, of course, the electric drill, with perhaps a magnetic grip, the pneumatic drill and the old-time ratchet brace. The last is often resorted to for site holes, especially when making connections to existing work.

The last three drills are small and portable, and have their own particular sphere of usefulness. Lack of space forbids a full description of these, and the student is referred to the makers' catalogues and the trade advertisements and articles in *The Engineer* and *Engineering*, etc.

Planing Machines are used for edge planing. Surface planing, as in bridge bearings, etc., is carried out on a surface-planing machine. Ordinary plates do not require to be surface-planed. In edge planing the tool travels, while the plate is at rest ; the reverse is the case with surface-planing machines. The usual maximum length of plate which edge-planing machines take is generally in the vicinity of 30 ft. Large machines can be obtained which can plane 40 ft. at one cut, which takes one minute. Most of the smaller machines, although not all, have open ends in the cast iron end frames or pillars. This permits of the edge planing of plates which are longer than the machine. When such long plates have to be edge-planed it necessitates a shifting of the plate to complete the machining, a process which means the use of the overhead travelling crane and all the resetting of the plate for alignment. The logical result is that firms try to arrange joints so that no plate is longer than the tool travel of their longest machine ; only one setting of the plate is therefore necessary.

Hand-riveting is used exclusively only in small structural yards or in an outside job where the small number of rivets does not warrant the use of a pneumatic outfit. The squad consists of four : two riveters, a holder-up, and the rivet-heater, who is generally a boy. The last-named heats the rivet until white-hot in an open top coke furnace with bellows underneath. This cylindrical furnace is easily transported, and stands about waist-high, with a diameter about half its height. The rivet is then thrown by the boy to the gang, who place it in the hole from the underside. The protruding shank is roughly "knobbed" down, and then finished off to shape with a snap die by the two riveters. The reaction to this hammering is provided on the underside by the holder-up and his dolly. The dolly is a long-handled hammer with a recessed head to fit round the original rivet head. A bent bar of iron hangs from the steelwork which is being riveted, and through the loop passes the shaft of the dolly. This distance, rivet to loop, is much less than loop to holder-up. The holder-up, by thrusting down his end of the handle, exerts a greatly intensified upward force on the rivet, and so counteracts the blows of the riveters.

Riveting, Pressure Machines.—The B.S.S. (2/5) states that "wherever possible the rivets shall be machine-driven, preferably by means of pressure machines of approved design." Two machines fulfilling the above condition are :—

Hydraulic Riveter.—High pressure water is permitted to enter a small cylinder by opening the inlet valve controlled by a lever. The water forces the ram out, and the rivet, coming between the stationary and the movable dies, is closed in one stroke.

The "holder-up" is stationary, and forms an integral part of the frame. The water in the cylinder is then released by moving the lever in the opposite direction and so opening the exhaust valve. On the cylinder emptying the ram returns to the commencement of its stroke.

Compressed Air or Pneumatic Riveter.—This machine is very similar to the hydraulic riveter in general outline, the power closing the rivet being obtained from compressed air in place of water. Messrs. De Bergue's 40 ton pressure machine for girder work requires only 4 cubic ft. of free air for each 1 in. diameter rivet. On a "straight run" 2,500 rivets, $\frac{3}{4}$ in. diameter, have been closed in a day by one machine, although a fair average is about 2,000.

Pneumatic Hammer, unlike the two previous machines, rains a series of blows, and from the noise it makes has earned the name of **Pom-pom.** It is virtually hand-riveting mechanised, since it is percussive and requires an external holder-up, pneumatic or otherwise.

The Pneumatic Chisel is mentioned here because it greatly esembles the pneumatic hammer. In this machine the small stroke nd rapidly acting piston, actuating the chisel, is also driven y compressed air.

Drifting.—It is emphasised in the B.S.S. (2/5) that bolts only hall be used for holding the plates together when riveting is in rogress. The only drifting permitted is the *gentle* pulling of the lates into position. The drift is a tapered bar, and by threading it through a series of nearly coincident holes and judiciously levering it the plate can be gently eased into position. This particular clause owes its origin to the practice of forcibly making holes register. By driving the tapered drift into a set of holes not quite coincident the edges of the holes are rounded and the circular holes become elongated. The finished rivet cannot therefore be up to the calculated strength.

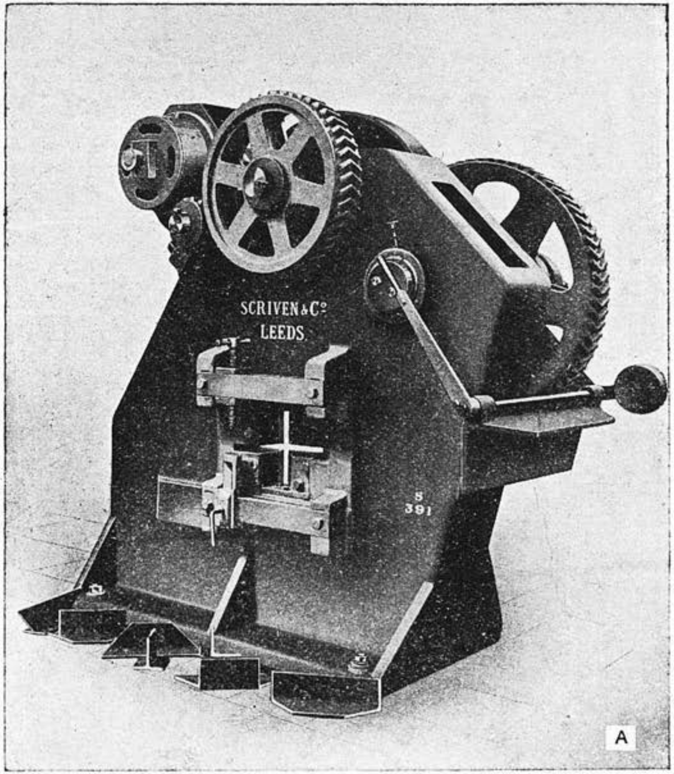
Painting.—Mill scale can only be removed by weathering or by chemical means. It is becoming more customary to specify that work shall be despatched to site unpainted. By this means most of the scale has been weathered off, and the steel can be safely painted at site.

Erection.—When the structure is to be sent abroad it is usual to specify that the ~~structure~~ whole, or part of it, should be erected in the yard of the constructional works. Thus, if the order is for several bridges all of the same span and not exceeding about 100 ft., one span at least is erected temporarily in the contractor's yard. The various elements composing the structure are then painted with a main position letter and a sub-position number, *e.g.*, G1 = Main Girder Diagonal No. 1, F3 = Floor Girder No. 3, etc. The general drawing is lettered to correspond with these markings. On occasion paints of different colours are used for coating the various parts of the structure. In the case of a shop an end bay and an intermediate one are usually erected and covered with the sheeting. Here again every element and sheet is lettered to facilitate the final erection abroad. The original estimate includes an item for this temporary work for inspection purposes.

For home jobs no erection is carried out in the yard of the constructional works unless for special reasons. The system of marking the parts is maintained, however, as it proves itself to be of great assistance to the erectors at the site.

Shipment.—On despatching the material abroad all sections are bundled together, since freight charges are based upon the overall bulk or the weight. There is one rate for volume and another for weight, and, needless to say, the higher of the two rates is charged. Thus similar gusset plates are piled together and bolted, while angles are nested into each other and wire-tied.

When the structure is to be erected at home the larger pieces are built up and riveted in the shops. The sizes of the pieces are limited by the transport facilities, the access to the site and the erecting equipment. Pieces which are exceedingly long or broad can only be sent by rail at a special rate, and only after having been inspected and accepted by the officials of the railway company. Such pieces are only sent when the traffic is light.



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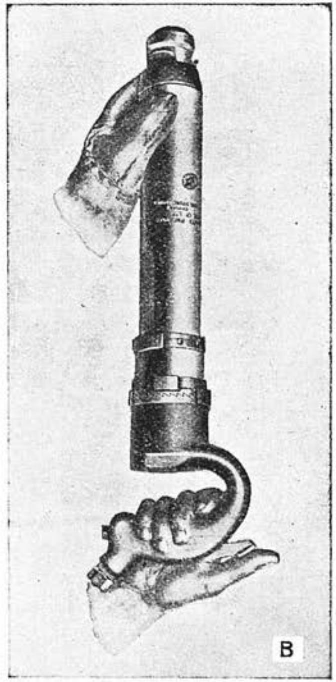
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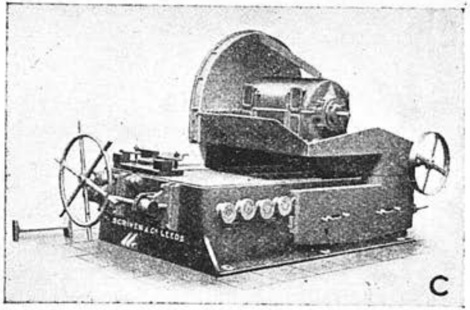
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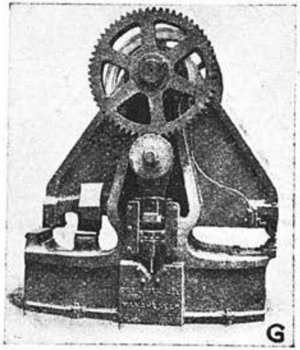
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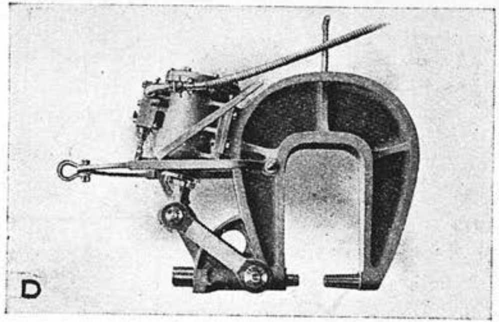
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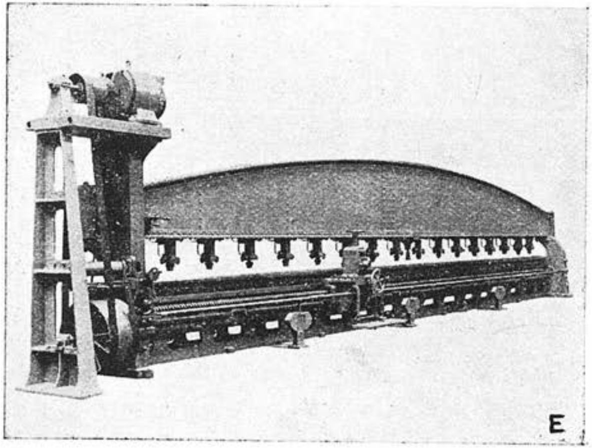
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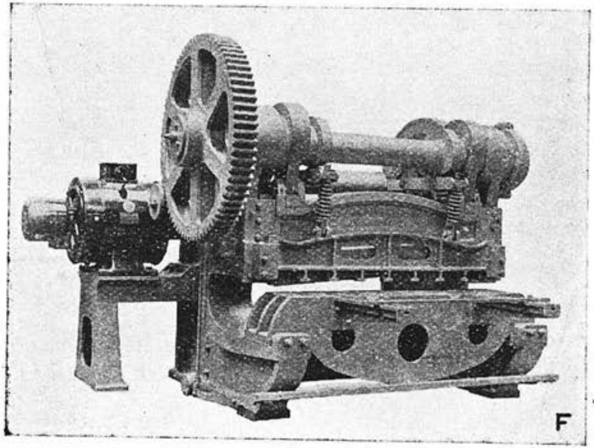
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ILLUSTRATIONS

A. *Angle and Tee Bar Cropping Machine.* (Scriven & Co., Ltd., Leeds.) Gives square or bevel cuts as shown by the specimens in the foreground.

B. *Pneumatic Riveting Hammer.* (Consolidated Pneumatic Tool Co., Ltd., London.) Piston diameter $1\frac{1}{16}$ in., stroke 4 in., weight 16 lb., 1,500 blows per minute. Overall length, $14\frac{3}{4}$ in.

C. *High-Speed Disc Saw.* (Scriven & Co., Ltd.) Cuts joists up to 18 in. × 7 in. cold.

D. *Pneumatic Riveting Machine.* (De Bergue & Co., Ltd., Manchester.) A portable machine with a $24\frac{1}{2}$ in. × 14 in. gap for rivets 1 in. diameter.

E. *Plate Edge Planing Machine.* (De Bergue & Co., Ltd.) Electrically driven 40-ft. planing machine with power-fed jacks.

F. 6 ft. × $\frac{3}{4}$ in. *Guillotine Shearing Machine.* (De Bergue & Co., Ltd.)

G. *Punching and Shearing Machine.* (De Bergue & Co., Ltd.) Will punch 1-in. diameter holes through and also shear 1 in. thick plates. Also cuts flats up to 6 in. × 1 in. and angles up to 6 in. × 6 in. × $\frac{5}{8}$ in.

CHAPTER IV

FASTENINGS, PITCHES AND SIMPLE RIVETED JOINTS OF PLATES

Fastenings.—Rivets and bolts are to steelwork what nails are to timber work. Rivets are the permanent fastenings; once completed they cannot be undone without considerable labour, accompanied by the total destruction of the rivet. Bolts and nuts are the temporary fastenings used in steelwork, although they are often adopted as permanent fastenings. A paragraph on their use will be found at the end of this chapter.

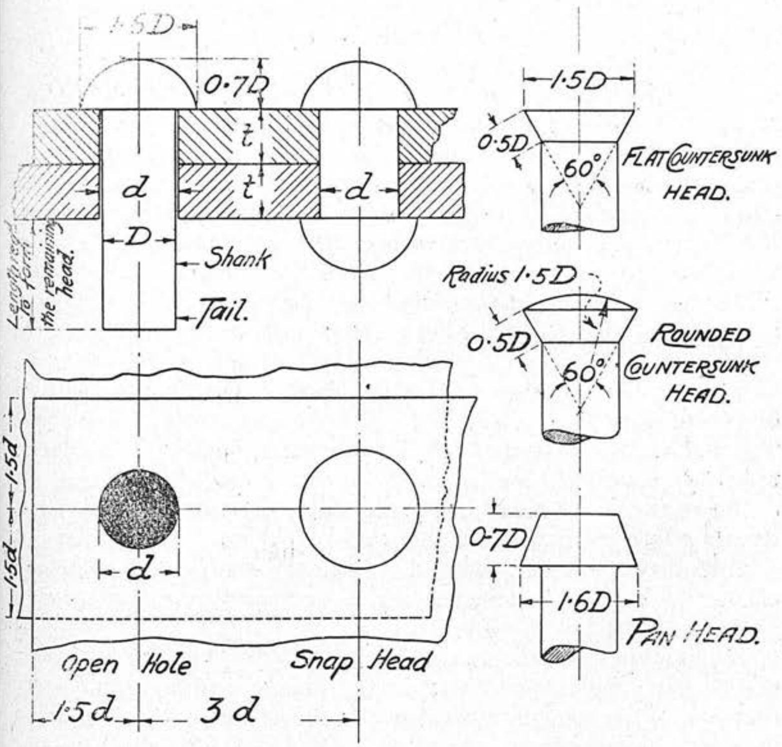
By aid of these fasteners rolled sections of all types are stitched together, and the strength of the resulting structure depends just as much upon the rivets and bolts as upon the rolled sections. If a well-designed structure be purposely overloaded so as to produce failure, it should have every unit and element in it failing simultaneously.

Rivet Material.—The B.S.S. (2/5) requires that rivets be made of steel of the standard specified. Wrought iron rivets were favoured by riveters and by some engineers for site work; at the site the facilities for riveting are never so good as in the shop, and wrought iron has the property of retaining its malleability after leaving the rivet forge or furnace longer than steel. Better formed rivet heads were consequently obtained when wrought iron was used. However, the use of wrought iron is not now countenanced and the contractor has to make the riveting up to standard with the stronger material.

Rivet Heads vary in size according to the different trades which use rivets, but even in structural work many forms of snap rivets can be found. At the time of writing several consulting engineers specify the dimensions of the rivet heads, and snaps to these dimensions have to be employed on all their work. Similarly, railway companies and constructional works show a preference for their own dies. Within certain limits the question of the dimensions of the heads for rivets in shear and bearing would seem to be one more of appearance than of strength.

Fig. 36 illustrates the rivet dimensions recently adopted by the British Engineering Standards Association in their *Standard*

Specification for Dimensions of Rivets ($\frac{1}{2}$ in. to $1\frac{3}{4}$ in. diameter), and the reader is referred to this publication (No. 275—1927) for further information. This specification, however, does not apply to boiler rivets, but represents a fair average of the sizes used at present in



D = The nominal diameter of the rivet, i.e. the dia. of the cold rivet before heating.
d = The diameter of the rivet hole = $D + \frac{1}{16}$"
The rivet strengths are calculated upon the finished or hole dia. d.

—BRITISH STANDARD RIVET HEADS.—

(Reproduced by permission of the British Engineering Standards Association)

FIG. 36.

British constructional work, and its publication will eventually secure uniformity in rivet dimensions.

The conical-headed rivet is practically obsolete in constructional work, and in consequence does not appear in the schedule, while the most common head of all is the snap or cup head. Where construction demands that the rivet head should not project overmuch the rounded countersunk head is used, and where an absolutely flush or level surface is required—as in the sole plate of a roof truss shoe, or in the bearings of the bridge illustrated in a succeeding chapter—one head is made flat countersunk. In the cases quoted the upper head is the snap and the lower one, in contact with the base plate, is the flat countersunk head, although occasions arise where it is necessary to have both heads countersunk, *e.g.*, in a bridge bearing where the upper rivet head fouls the bottom end of a tightly fitted stiffener.

Rivet Diameter.—The hole in the plates is $\frac{1}{16}$ in. greater in diameter than the cold rivet shank previous to riveting. This clearance permits of the easy entry of the hot rivet shank into the hole. Just sufficient clearance should be given because the rivet, when being riveted under pressure, should squelch up and completely fill the hole. When the shank cools it contracts both in length and in diameter. The cold rivet heads therefore pull the plates together, while the shank cannot theoretically completely fill the hole. The larger the initial clearance the more difficult is the task of obtaining an ideal finished rivet which will completely fill the hole. The B.S.S. (5/2) says that the initial clearance shall not be larger than $\frac{1}{16}$ in. nor less than $\frac{1}{32}$ in.

Owing to this clearance, ambiguity has long existed as to what a drawing specifying $\frac{3}{4}$ in. diameter rivets meant. Did it mean a $\frac{3}{4}$ in. diameter hole and the cold diameter of the shank previous to closing $\frac{11}{16}$ in., or did it mean $\frac{3}{4}$ in. diameter of cold shank with a $\frac{13}{16}$ in. diameter hole ?

The new specification (B.S.S. (2/4) and (4/23)) has now definitely settled this point by stating that the diameter of the rivet is the diameter of the hole (or finished rivet), and all calculations are based upon this diameter.

Nominal Diameter is not the real diameter, but the diameter of the cold shank of the rivet previous to heating. American specifications work to this diameter, and all their rivet values are based upon it.

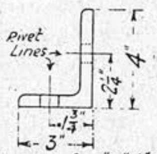
Rivet Grip and Ordering of Rivets.—The grip of the rivet is the distance from the underside of one head to the underside of the remaining head of the completed rivet.

The length of the rivet as ordered from the makers is the grip plus the extra length required to form the second head.

There is always a certain amount of extra thickness, or "gather," where several plates are clenched by the same rivet, since plates do not lie mathematically flat against each other. In the case of long grip rivets, therefore, the ordered length is obtained by making an additional allowance which roughly increases with the number of plates. All constructional yards have their own lists giving the order length for a stated grip. The ordering of rivets only affects the works, and not the design.

Rivets closed by hand do not require the same length of shank as machine-closed rivets, and are ordered from about $\frac{1}{8}$ in. to $\frac{1}{4}$ in. less in length. The machine-closed rivet therefore fills the hole in a more satisfactory manner. If the length, underside of head to tail or point of rivet shank, is too great, the rivet head when completed has a collar round its base; in appearance it may be likened to a washer fastened under the cup head. On the other hand, if the shank is too short an incomplete rivet head is formed. The rivet heads should bear tightly on the surface of the plate and otherwise fully meet the B.S.S. (2/5).

Rivet Line, also known as the scrieve line and in America as the gauge or gage line, is the imaginary line along which rivets are placed. If the angle has equal legs, then the rivet lines are at equal distances from the heel, while if the legs be unequal, so also are the distances of the rivet lines from the heel, as in Fig. 37. Practice has assigned to each section rolled "standard" positions of rivet lines, which are conformed to wherever possible. The positions of these lines depend upon the flange or leg width, and are independent of the thicknesses of the bars. In the case of very thick angle bars little room is left for the closing of the rivet head by the adjoining leg of the angle (see p. 7). In these circumstances departure from the "standard" rivet line is sometimes necessary if the rivet diameter is to be maintained; alternatively a smaller diameter of rivet may be chosen in conjunction with the standard rivet line. The thickness of the bar, therefore, may affect the diameter of the rivet to be used. It is good practice always to dimension the position of the rivet line whether it is standard or not, as this eliminates errors both in the drawing office and in the works. In channel flanges the same rules are followed as for the legs of angles, while the flanges of joists are holed similarly to the tables of tee bars (see illustration to Table 8, Vol. III.).



Rivet Lines for 4"×3"×½" L

FIG. 37.

Mnemonic for Angle and Channel Rivet Lines.—To find the distance of the rivet line of an angle from the heel of the bar, add $\frac{1}{2}$ in. to the leg width and divide the total by 2. This rule holds good for angles of $2\frac{3}{4}$ in. leg and over. For angles of $2\frac{1}{2}$ in. leg and under substitute $\frac{1}{4}$ in. for the $\frac{1}{2}$ in. mentioned. In Fig. 37, since the bottom leg is 3 in., the rivet line dimension is $(3'' + \frac{1}{2}'') \div 2 = 1\frac{3}{4}''$; similarly for the 4 in. leg the dimension is $2\frac{1}{4}$ in. For a $2\frac{1}{2}$ in.

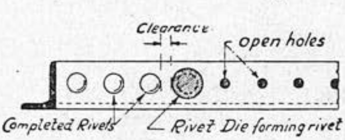
leg angle the dimension to the rivet line is $(2\frac{1}{2}'' + \frac{1}{4}'') \div 2 = 1\frac{3}{8}''$. These rules are also applicable to channels whose flanges have the same widths as the angle legs mentioned, and also to the stems of tees, the depth of the stem being equivalent to an angle leg.

Pitch.—The pitch is the distance from centre to centre of adjoining rivets lying on the same rivet line. The pitch is kept constant over as large a length as is possible, and is not varied from rivet to rivet. It is better to have a large number of common pitches with an odd pitch cast to each end than to have all the pitches the same with each pitch containing, say, an odd $\frac{1}{16}$ in.

E.g., 1 pitch @ $3\frac{1}{8}''$ + 20 intermediate pitches of $3''$
+ 1 end pitch @ $3\frac{1}{8}'' = 5'\ 6\frac{1}{4}''$ total

is better than 20 pitches @ $3\frac{5}{16}''$ $= 5'\ 6\frac{1}{4}''$ total

as it causes the template-maker less trouble to mark off; besides, it is easier to check the job and thus keep down errors. The time saved by the use of an easy pitch of 3 in. will more than pay for the two extra holes required.



Minimum Pitch
FIG. 38

The template-maker has often template strips marked and holed to the standard pitches of 3 in., $3\frac{1}{2}$ in. and 4 in. By setting off the two odd pitches first the intermediate pitches are filled in at once by using the appropriate standard holed strip.

In roof trusses $2\frac{1}{2}$ in. and $2\frac{3}{4}$ in. are common pitches with $\frac{3}{4}$ in. diameter rivets. With plate web girders the most common and desirable pitches are 3 in., $3\frac{1}{2}$ in., 4 in., $4\frac{1}{2}$ in., 5 in. and 6 in. Pitches like $2\frac{3}{4}$ in., $3\frac{1}{4}$ in., $3\frac{3}{4}$ in., etc., are permissible, but odd sizes like $3\frac{1}{16}$ in., $3\frac{3}{16}$ in., etc., should be eliminated where possible. The pitch multiplication table given in Table 9, Vol. III., will be found useful, especially in plated work.

Minimum Pitch is specified in the B.S.S. (4/25) to be three times the diameter of the finished rivet. If the rivets are spaced closer than this, very little clearance is left between the last-formed rivet head and the die forming the next rivet head. From Fig. 38 it will be seen that the absolute minimum pitch is half the diameter of the die plus half the diameter of the adjoining finished rivet head provided that the rivet head is in true position. Stating this as a formula, the absolute minimum pitch is roughly 2 d. $+ \frac{1}{2}''$, where d. = the diameter of the hole.

Where an angle has rivets through both legs care has to be exercised that proper clearance is given for riveting. In Fig. 39

the rivets are reeled; that is, the rivets in the vertical leg come midway between those in the horizontal leg. In Fig. 40 the rivets in the vertical and horizontal legs occur at the same cross-section. The $4'' \times 3'' \times \frac{3}{8}''$ angle of Fig. 41 illustrates the problem of riveting which occurs in Fig. 40. Clearance is given to the die when closing rivet B if rivet A has been closed first. On the other hand, if rivet B be closed first, no clearance is given to close rivet A; the position of the die for this case is shown in broken and slightly hatched line. The choice of the size of the angle and of the rivet diameter requires a little care on the beginner's part in order that the rivet heads do not foul.

Further, there is the weakening of the angle, due to rivet holes, to be considered. In the cross-section of the angle with reeled holes, Fig. 39, there is one rivet hole out, whereas in that of Figs. 40

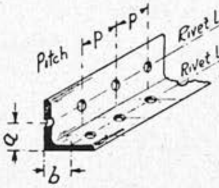


FIG 39.

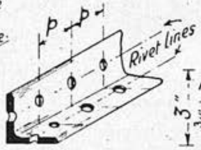


FIG. 40.

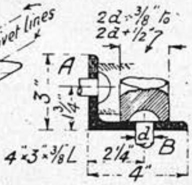


FIG. 41

and 41 there are two. It is, therefore, the practice that wherever possible rivet holes are reeled as in Fig. 39.

In Fig. 43 clearance is obtained for riveting by reeling the rivets, and the section is weakened by two rivet holes, not four, at any cross-section. It would be impossible in this case to have four rivets, all 1 in. in diameter, placed on the same cross-sectional line after the manner of Fig. 40.

Reeled Pitch, Alternate Pitch or **Staggered Pitch** occurs when an angle has a double rivet line on one or both legs, as in Figs. 42 and 43. The distance measured along one rivet line from the centre of a rivet on it to the centre of the adjoining rivet on the lower and parallel rivet line is known as the alternate pitch, reeled pitch or staggered pitch. This is in contradistinction to the word "pitch," which means from rivet to rivet on the same line.

Ambiguity may arise as to what a drawing means by the word "pitch" in the case of an angle having a double rivet line. The method of stating the pitch employed in Fig. 43*a* is to be preferred to that of Fig. 42*a*. There is not much to choose between the two methods, however, if the drawing is dimensioned to show what is actually meant.

When the angle has two 6 in. legs the riveting is reeled after the manner of Fig. 43. One rivet is near the angle root, while the other rivet, on the same cross-sectional line of the angle, is nearer the other toe. If the pitch is wide, the angle has only two rivet holes taken out of its gross cross-sectional area.

Where one leg is 6 in. and the other is $3\frac{1}{2}$ in. or so, the method of Fig. 42*a* and *b* is adopted. The $3\frac{1}{2}$ in. leg being approximately half

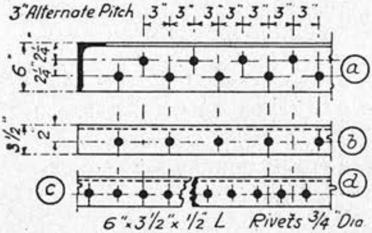


FIG. 42

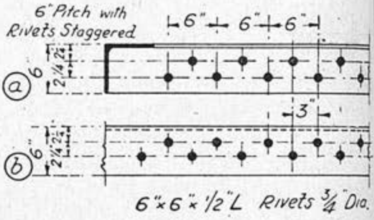
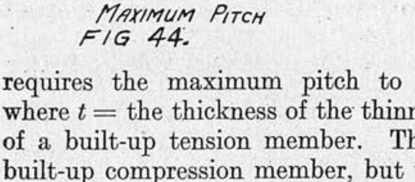


FIG. 43

the width of the 6 in. leg, will be half as strong, and will therefore only require half the number of rivets. However, should it be desired to have a 3 in. pitch on the $3\frac{1}{2}$ in. leg, either system, as indicated by views **c** and **d**, may be used. Method **c** cannot be used if the rivets are large, because two rivets will occur on the same cross-sectional line close to the root of the angle, and will bring up the difficulty of clearance for riveting, as illustrated in Fig. 41. System **d** could be adopted where the rivet diameter is large. Method **d** is not necessarily more economical in providing net cross-sectional area than method **c**; see the B.S.S. rules on diagonal tearing as discussed in the first article of Chapter V.



MAXIMUM PITCH
FIG 44.

Maximum Pitch (Fig. 44).—If the rivets be widely spaced or pitched, the plates, especially if thin, are apt to gape apart. The B.S.S. (4/26) requires the maximum pitch to be not greater than "16 t," where $t =$ the thickness of the thinnest plate or section in the case of a built-up tension member. The same reasoning applies to a built-up compression member, but with the additional question of buckling. The various plates of a compression member, *e.g.*, a column, strut, or the top flange of an ordinary plate web girder, become little individual columns held to the neighbouring plates by the rivet heads. The rivet heads are under tension, and the wider the spacing the greater is the load on the rivet head, tending to break it away from the shank. Again, the further apart the rivets are the more flexible become the little columns, so that should one rivet head give way an individual column of twice the ordinary length occurs. This may finally lead to the entire failure of the built-up member. The maximum pitch in the case of a built-up compression member is limited to "12 t."

Calculation in the case of a plate web girder may, for strength, only require a 5 in. pitch, but 12 t, if $t = \frac{1}{4}$ in., would demand a 3 in. pitch. The extra rivets are added to stitch or tack the plates together.

The same reasoning applies to a reinforced concrete column where the links or rodding of the column are never spaced further apart than sixteen times the diameter of the thinnest vertical bar.

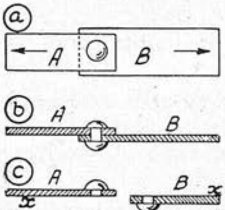
Single Shear.—Fig. 45 shows two plates being pulled apart, the upper plate, A, to the left, and the lower plate, B, to the right. If the rivet is weak, it will fail by shearing along the plane xx, the surface common to both plates. The area of metal sheared through is the circular cross-sectional area of the rivet shank, *i.e.*, an area equal to the plan area of the hole. If f_s be the safe working single shear stress in tons per square inch, then one rivet will safely carry a load of $f_s \times$ area of hole $= f_s \times \frac{\pi d^2}{4}$ tons $= 0{\cdot}7854\ d^2 f_s$, where $d =$ diameter of the hole in inches, and N times this if there are N rivets.

Double Shear can only occur when there are at least three plates or members riveted together with a common rivet. Referring to Fig. 46, the plate B will travel to the right, on the failure of the rivet, with a piece of the rivet shank embedded in the hole. The rivet has thus been sheared at two sections: xx and yy. Since there are two single shears brought into action simultaneously, this is known as double shear. The B.S.S. (4/23) assumes double shear to be twice as strong as single shear.

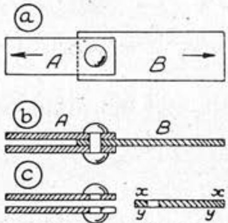
If f_{ds} be the safe working double shear stress in tons per square inch, then the load which one rivet will safely carry is $f_{ds} \times$ area of finished shank or hole $= f_{ds} \times \frac{\pi d^2}{4}$ tons $= 0{\cdot}7854\ d^2 f_{ds}$. Alternatively, since there are two single shears, then the safe load per rivet in tons $= 2 f_s \times 0{\cdot}7854\ d^2 = f_{ds} \times 0{\cdot}7854\ d^2$, where d is the diameter of the hole in inches.

Crushing or Bearing of a Rivet or Plate.—If a cylindrical bar of steel be laid lengthwise on some plastic material (like putty), it will gradually sink into the mass. Place the rod vertically with one end resting on the plastic substance, and its rate and depth of penetration will be greater in this case than in the previous one.

The weight of the rod is constant, so that penetration must be a function of the areas in contact. It has been found experimentally that the bearing resistance varies directly as the *projected area of*



SINGLE SHEAR. FIG. 45



DOUBLE SHEAR

FIG. 46.

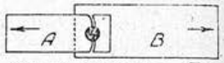


PLATE TEARING FIG. 47

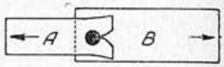
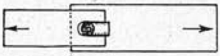
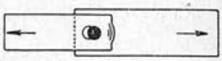


PLATE BURSTING FIG. 48.



PIECE SHEARING OUT OF PLATE FIG. 49.

— DIFFERENT WAYS BY WHICH —
— A RIVETED JOINT MAY FAIL. —



CRUSHING OF PLATE FIG. 50.

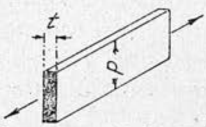
contact at right angles to the direction of motion. In case 1 this was a rectangle equal to the length of rod by the diameter of rod; in case 2 the area was a circle equal to the cross-sectional area of the rod.

Fig. 50 indicates the rivet crushing the plate; similarly if the plate were the harder material it would crush the rivet. Crushing and shearing of a rivet are thus quite different.

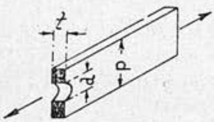
If f_b = permissible bearing stress in tons per square inch, then the value of a rivet of diameter d bearing on a plate of thickness $t = f_b \times d \times t$ tons.

The bearing value of a rivet can therefore be raised either by thickening the plate or choosing a larger diameter of rivet or by increasing both.

When calculating the bearing value of a rivet with a countersunk head only one-half the depth of the C'S'K portion is to be considered as effective (B.S.S. (4/23)); the diameter to calculate on is the ordinary rivet diameter.



GROSS AREA = pt
FIG 51.



NET AREA = $(p-d)t$
FIG. 52.

Failure of Riveted Joints.—(**1**) Plate tearing across at the weakest section, *i.e.*, where there is least metal, as at a rivet hole (Fig. 47).

In Fig. 51 the whole cross-sectional area can be utilised to carry a tensile stress or pull. If f_t = permissible tensile stress per square inch, then the permissible load it can carry previous to holing is $f_t \times p \times t$.

The safe tensile load after holing (Fig. 52) $= f_t \times (p - d)\, t$, since only the **Net Area** can carry a load. To approximate to the net area of a plate with a C'S'K hole in it, the B.S.S. (4/24) assumes the hole to be $\frac{1}{8}$ in. larger in diameter than the actual diameter of the rivet.

2. The joint may give way, due to the single or double shearing of the rivet (Figs. 45 and 46).

3. Failure may occur through crushing, as previously explained (Fig. 50).

4. The plate may rupture or burst in front of the rivet, as in Fig. 48.

5. Finally, the end piece of plate may be sheared out of the plate (Fig. 49).

Edge Distance and Margin.—It is obvious that the nearer the hole is to the edge of the plate the greater is the chance of failure by either 4 or 5, as explained above. Experience has shown that if the margin between the edge of hole and edge of plate is at least equal to the diameter of the rivet, failure does not occur. Or, as shown in Fig. 36, the edge distance from the centre of the hole to the edge of plate is equal to $1\frac{1}{2}d$ (as a minimum), where $d =$ the diameter of the hole.

In the case of sheared edges, where the metal may have suffered injury due to being sheared, the edge distance is increased to $1\frac{3}{4}d$ (B.S.S. (4/27)).

Thus, if the rule for edge distance is followed, failure can only occur by ~~either~~ one of the ways mentioned in **1**, **2** or **3** or any combination of these three.

Efficiency of a Joint is the ratio of the least strength of a joint—*i.e.*, by **1**, **2** or **3**—to the strength of the original plate previous to being holed. This fraction or ratio is usually multiplied by 100, and is thus expressed as a percentage.

Long Grip Rivets.—When the grip of the rivet becomes long the rivet is subjected to bending in addition to bearing and shearing stresses. To allow for this extra stress on the rivets, the American specifications state that rivets carrying calculated stress, and whose grip exceeds 4 diameters in length, shall be increased in number by 1 per cent. for each additional $\frac{1}{16}$ in. grip. The grip has, therefore, an important bearing upon the fixing of the rivet diameter.

Rivets through Packings.—The beam action of the rivet and its accompanying bending action are perhaps more apparent in this case. See Fig. 53. Where the packings exceed $\frac{3}{8}$ in. in thickness (B.S.S. (4/28)) the number of rivets required shall be increased by at least 20 per cent. of the required net number. The additional rivets are to be placed outside one of the connected members after the fashion of Fig. 54. Some of the load of the two lower main bars is picked up by the bottom rivets and transferred into the lengthened packer, and ultimately into the two outside upper plates. This lengthened packing is, in effect, equivalent to the swelling up of the two centre plates into a large "monolithic" head held between the two outside top plates. The stress in the two main lower plates "percolates" through to the packings, and is distributed approximately evenly throughout this head (Fig. 55).

The rivets on cooling contract and grip the plates tightly together. Due to this, a frictional resistance is set up between the various plate surfaces, resisting motion when the ends of the plates are pulled, thus reducing shear.

That portion of the joint designated as the massive head is more monolithic than any other portion, since it has the greatest number

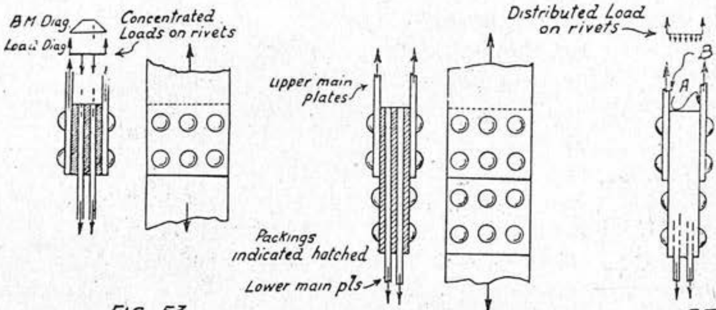


FIG. 53. FIG. 54. FIG. 55.

of rivets through it. When frictional resistance fails it will do so on the inner faces, A and B, of the two outer main plates, and direct single shear will be set up at both these faces, accompanied, of course, by bearing. The bending action on the upper rivets, therefore, is reduced considerably. In fact, if the packers are carried sufficiently far down and attached by a corresponding number of extra rivets to the two mid-plates, the bending action on the upper rivets will be entirely eliminated. This assumes, naturally, that every rivet completely fills the hole through which it passes, and that its load = the total load ÷ the number of rivets through the riveted member.

Working Stresses.—It is found that the shear, crushing and tensile strengths bear a more or less definite relationship to each other, and so it is common usage to express the shear and bearing strengths in terms of the tensile strength.

From his study in "Strength of Materials" the student will be conversant with the fact that it is not the ultimate tensile strength (of say 30 tons per square inch) which is used in designing, but a lesser figure in order to be safe. The reduction factor is known as the **Factor of Safety** (say 4, giving a working or permissible stress of $30 \div 4 = 7{\cdot}5$ tons per square inch).

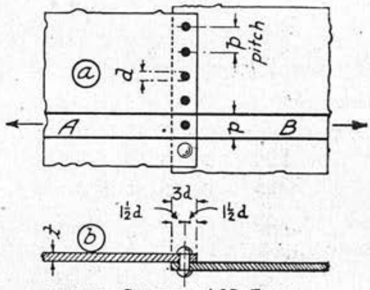
$\mathbf{f}_t$ = permissible tensile stress in tons per square inch on net area = ultimate stress ÷ factor of safety.

$\mathbf{f}_b$ = permissible bearing or cushing stress in tons per square inch $\Big\} = 1\frac{1}{2}\,\mathbf{f}_t$.

$\mathbf{f}_s$ = permissible single shear stress in tons per square inch $= \frac{3}{4}\,\mathbf{f}_t$.

$\mathbf{f}_{ds}$= permissible double shear stress in tons per square inch of rivet shank cross-section $= 2\mathbf{f}_s$ (see B.S.S. (4/23)) $\Bigg\} = 1\frac{1}{2}\,\mathbf{f}_t$.

SINGLE-RIVETED LAP JOINT.—When working out efficiencies it is immaterial from a strength point of view whether a pitch width or a 20 ft. width of plate be considered; what applies to a pitch width applies to any multiple of the pitch. Consider Fig. 56, where the plates have a single row of rivets and are lapped, by the foregoing rules, $1\frac{1}{2}d$ on each side of the rivet centre line, viz., $3d$ total. Take that portion AB of the plates shown in heavy line, which is a pitch in width.



SINGLE RIVETED LAP JOINT
FIG. 56.

Plate Efficiency : $\dfrac{(p-d)t \times f_t}{ptf_t}$

$$= \frac{\text{tensile value of plate after holing}}{\text{tensile value of plate previous to holing}} = \frac{p-d}{p} \quad . \quad \mathbf{1}$$

Shear Efficiency : $\dfrac{0{\cdot}7854d^2 \times f_s}{ptf_t}$

$$= \frac{\text{single shear value of one rivet}}{\text{tensile value of plate previous to holing}} = \frac{0{\cdot}589d^2}{pt} \quad . \quad \mathbf{2}$$

since $f_s = \frac{3}{4}f_t$.

Bearing Efficiency : $\dfrac{dtf_b}{ptf_t}$

$$= \frac{\text{value of one rivet in bearing}}{\text{tensile value of plate previous to holing}} = \frac{1{\cdot}5d}{p} \quad . \quad \mathbf{3}$$

since $f_b = \frac{3}{2}f_t$.

Joint Efficiency is the lowest value of equations **1**, **2** and **3** . **4**

Numerical Example.

Let $d = \frac{3}{4}''$ finished diameter, $p = 3d = 2\frac{1}{4}''$ and $t = \frac{3}{8}''$.

Plate Efficiency :

$$\frac{p-d}{p} = \frac{2\frac{1}{4} - \frac{3}{4}}{2\frac{1}{4}} = \tfrac{2}{3} \text{ or as a percentage} = 66\tfrac{2}{3} \text{ per cent.}$$

Shear Efficiency :

$$\frac{0{\cdot}589d^2}{pt} = \frac{0{\cdot}589 \times (\frac{3}{4})^2}{2\frac{1}{4} \times \frac{3}{8}} = 0{\cdot}393 \text{ or as a percentage} = 39{\cdot}3 \text{ per cent.}$$

Bearing Efficiency :

$$\frac{1{\cdot}5d}{p} = \frac{1{\cdot}5 \times \frac{3}{4}}{2\frac{1}{4}} = 0{\cdot}5 \text{ or as a percentage} = 50 \text{ per cent.}$$

Joint Efficiency is thus the shear efficiency $= 39{\cdot}3$ per cent.

By raising the rivet diameter the shear area is increased, and therefore so also is the shear efficiency. Using a $\frac{7}{8}$-in. diameter rivet at the minimum pitch of $3d = 2\frac{5}{8}''$, the efficiencies are respectively, $66\frac{2}{3}$ per cent., 45·8 per cent. and 50 per cent. The question now arises as to what diameter of rivet gives maximum efficiency.

The joint should be equally strong as regards plate tearing, rivet shearing and failure in bearing, *i.e.*,

$$(p-d)tf_t = 0{\cdot}7854d^2f_s = dtf_b \quad . \quad . \quad . \quad . \quad . \quad \mathbf{5}$$

From the first two of **5**, *i.e.*, $\mathbf{1} = \mathbf{2}$, $p - d = 0{\cdot}589d^2 \div t$

$$\text{or } p = (0{\cdot}589d^2 \div t) + d \quad . \quad . \quad . \quad . \quad . \quad \mathbf{6}$$

$$\mathbf{1} = \mathbf{3} \quad p - d = 1{\cdot}5d, \text{ whence } p = 2{\cdot}5d \quad . \quad . \quad . \quad . \quad . \quad \mathbf{7}$$

$$\mathbf{2} = \mathbf{3} \quad 0{\cdot}589d^2 \div t \doteq 1{\cdot}5d, \text{ therefore } d = 2{\cdot}54t \quad . \quad . \quad . \quad . \quad \mathbf{8}$$

From **8** the rivet diameter is for $\frac{3}{8}$-in. plates $= 2{\cdot}54 \times \frac{3}{8}'' = 0{\cdot}95''$, so adopt 1-in. diameter rivets. The pitch given by **6** is 2·57″ and by **7** is 2·5″, both of which are impossible because of the $3d$ rule, so that the pitch to use must be the smallest possible, namely, $3d$ or 3 in. The respective efficiencies are now $66\frac{2}{3}$ per cent., 52·4 per cent. and 50 per cent.

Analysing **6**, **7** and **8** on the supposition that t, the plate thickness, is 1 in., it follows by **8** that the rivet diameter $d = 2{\cdot}54''$, which is too large for the satisfactory closing of the second and final head of the rivet.

Equation **7** is eliminated by the rule which states that the minimum pitch is limited to 3 diameters. It is generally impossible to arrange a joint so that its strength is equal in all directions.

If the minimum pitch is limited to 3 diameters, then the efficiency for this type of joint can never rise above 50 per cent. Thus with $3d$ pitch the efficiencies of **1** and **3** are $66\frac{2}{3}$ per cent. and 50 per cent., and with a pitch of $4d$ they are respectively 75 per cent. and 37·5 per cent. The economical pitch of $2\frac{1}{2}d$ is too close for ordinary work.

Rules for Rivet Diameters.—From the above it will be seen that the theoretical size of rivet cannot be used because of practical considerations. Where a rolled section, such as an angle, is to be riveted to a plate, the position of the rivet line ordains the maximum size of rivet possible, because of the necessary clearance required by the die next the adjoining leg. Thus a $\frac{3}{4}$-in. diameter rivet is the largest which can be used in a $2\frac{1}{2}$-in. angle leg.

The rivet grip should not be over-long and the rule for grip of 4 diameters should be borne in mind. If the grip is long, then the diameter should be proportionately larger.

Again, in a girder or roof truss, sometimes even throughout a whole job if small, the rivet diameter is kept constant to save changing of drills, and also to save different sizes of rivets at site. As a rough guide the following approximate classification is given.

About $\frac{1}{2}$ in. dia.	Reserved for very light work, such as tanks, light steel doors, etc.
,, $\frac{5}{8}$,,	Light roof trusses of small span, say up to 25 ft.
,, $\frac{3}{4}$,,	Roof trusses, light lattice and plate web girders.
,, $\frac{7}{8}$,,	Large-span roof trusses (100 ft., say) and medium to heavy lattice and plate girders.
,, 1 ,,	For heavy work, *e.g.*, 80-ft. span double-track railway bridge.
,, $1\frac{1}{8}$,,	Exceptionally heavy work, and is employed only in the special parts requiring the heavy rivets, and if possible not throughout the whole structure.
,, $1\frac{1}{4}$,,	Have been used in constructional steelwork but seldom, and only in such structures as Hell Gate Arch Bridge, etc.

No one can be dogmatic on the classification of rivet diameters, as every job must be considered, more or less, on its own merits.

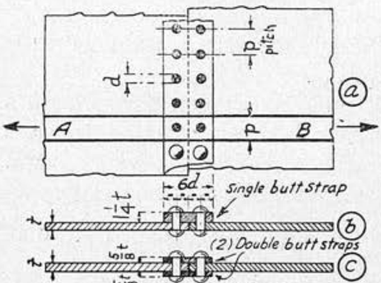
The student will, however, be safe in limiting himself to the choice of rivet diameters of approximately $\frac{3}{4}$ in. to $\frac{7}{8}$ in. When he has to design heavy structures he will be awarded the job because of the extent of his knowledge, and will settle the size of the rivet in most cases apparently without thought.

The following are two well-known rules for plate joints, but are not of much use in constructional steelwork.

Unwin.—$d = 1{\cdot}2\sqrt{t}$ to $1{\cdot}4\sqrt{t}$, where $t =$ plate thickness and $d =$ the rivet diameter.

Another rule is, "d shall not be less than t."

Single-riveted Butt Strap Joint, Fig. 57.—The two original plates are made to butt, while the covers are placed over and under the joint as in c, or a thicker cover over one face of the joint as in b. General practice gives the covers the thicknesses shown, so that if failure occurs by tearing of plate, it will be the main plate which tears. The width of the covers must be at least $6d$ in order to give an edge distance of $1\frac{1}{2}d$ to all four edges of plates and covers.



SINGLE RIVETED BUTT JOINT.
FIG 57.

In b the rivet is in single shear with minimum bearing on the main plate.

In c the rivet is in double shear with minimum bearing on the main plate, the two covers presenting a greater bearing thickness than the main plate. (Fig. 46 illustrates this point as well as double shear.)

Numerical Example.—Fig. 57 a and c, *i.e.*, double covers. Main plate $\frac{3}{8}$ in. and minimum pitch limited to $3d$. To design the joint for maximum efficiency:—

Plate Efficiency: $$\frac{(p-d)tf_t}{ptf_t} = \frac{p-d}{p} \quad . \quad . \quad . \quad . \quad . \quad . \quad 9$$

Shear Efficiency: $$\frac{0{\cdot}7854d^2f_{ds}}{ptf_t} = \frac{1{\cdot}1781d^2}{pt}, \text{ since } f_{ds} = \tfrac{3}{2}f_t \quad . \quad 10$$

Bearing Efficiency: $$\frac{dtf_b}{ptf_t} = \frac{1{\cdot}5d}{p}, \text{ since } f_b = \tfrac{3}{2}f_t \quad . \quad . \quad 11$$

For all-round maximum efficiency **9** = **10** = **11**.

9 = **10** $\quad p - d = 1{\cdot}1781d^2 \div t,$

whence $p = (1{\cdot}1781d^2 \div t) + d$ **11a**

9 = **11** $\quad p - d = 1{\cdot}5d$, whence $p = 2{\cdot}5d$ **11b**

10 = **11** $\quad \dfrac{1{\cdot}1781d^2}{t} = 1{\cdot}5d$, whence $d = 1{\cdot}2732t$ **11c**

The rivet diameter is fixed by **11c** as being $1{\cdot}2732 \times \frac{3}{8} = 0{\cdot}48''$; adopt $\frac{1}{2}$ in. The minimum pitch of $3d$ supersedes equation **11b**, so that the pitch used must be $1\frac{1}{2}$ in. The respective efficiencies of **9**, **10** and **11** are now $66\frac{2}{3}$ per cent., $52{\cdot}4$ per cent. and 50 per cent.

Maximum Efficiency of Single Riveted Joints.—So long as the minimum pitch is limited to 3 diameters, the joint efficiency can never exceed 50 per cent. by equation **11**.

Diagonal Pitch and Distance apart of Rivet Lines.—The distance between rivet centres, whether on the same rivet line or not, is limited to a minimum of $3d$ (by specification) for riveting. The absolute minimum may be slightly less, as explained in the text referring to Fig. 38, but is only used in exceptional circumstances.

In Fig. 58c the three rivets C, D and E form the corners of an equilateral triangle, where each side is of the minimum length $\mathbf{3d}$ = pitch $\mathbf{p}$. The distance $\mathbf{e}$ between the rivet lines is thus limited to a minimum of $\frac{\sqrt{3}}{2}\mathbf{p} = 0{\cdot}866\,\mathbf{p}$, so that if $\mathbf{e}$ is made $\frac{7}{8}\mathbf{p}$ ($= 0{\cdot}875\,\mathbf{p}$), the diagonal distance C D is at the least $\mathbf{3d}$.

The diagonal distance is never given on a drawing, as it affords no help to the template maker, who is interested only in the "rectangular co-ordinates" of the rivet centres, viz., the edge distances, distance between rivet lines, and the pitch. If the designer makes $e = \frac{7}{8}p$, or (as is usual in structural work) $= p$, he knows that the diagonal distance is more than $3d$ and that easy riveting is possible.

In Figs. 42 and 43 the pitch is 6 in. in the 6-in. legs, and therefore the distance e apart of the rivet lines can, from the point of view of riveting, be closer than $\frac{7}{8}$ of $3d$. This at once follows, because the 6 in. pitch is not the minimum pitch of $3d$. If the rivets had been 1 in. diameter the distance apart of the rivet lines, although only $2\frac{1}{4}$ in., would be still satisfactory.

Considering one pitch width of plate, shown shaded in Fig. C, and forgetting the existence of the plate on either side of it, there is the possibility of the strip tearing—

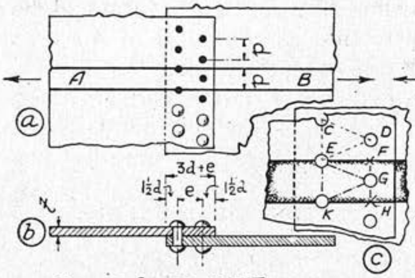
(a) along lines EK or FH net length $= (p - d) = 2d$.
(b) ,, ,, EGH ,, $= 4\frac{1}{2}d - \frac{3}{2}d = 3d$.
(c) ,, ,, EGK ,, $= 6d - \frac{4}{2}d = 4d$.

The B.S.S. (4/4) states that if there is the possibility of a plate tearing diagonally, then only $\frac{4}{5}$ of the net diagonal length shall be considered as being effective. Taking this into consideration, (b) and (c) are still much longer than (a), and, therefore, in the event of failure by tearing it will be straight across from E to K, or F to H, as in our former calculations.

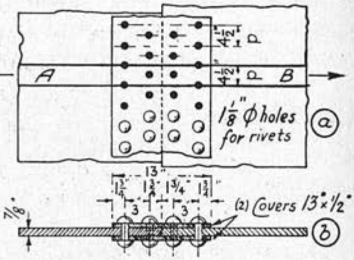
The question of zigzag tearing is dealt with more fully in the opening article of Chapter V.

Double-riveted Lap Joint.—From Fig. 58 it will be seen that a pitch width of plate contains two rivets, viz., two halves and one complete rivet. The shear and bearing equations will therefore have the coefficient **2** in the numerator.

Numerical Example.—Design a double-riveted lap joint for two $\frac{3}{4}$-in. thick plates so that the joint may be as strong as possible;



DOUBLE RIVETED LAP JOINT.
FIG 58.



DOUBLE RIVETED, DOUBLE BUTT STRAP JOINT.
FIG. 59.

minimum pitch is limited to $3d$, and the diameter of the rivet must not exceed 1 in.

Plate Efficiency: $$\frac{(p-d)tf_t}{ptf_t} = \frac{p-d}{p} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad \mathbf{12}$$

Shear Efficiency:

$$\frac{2 \times 0{\cdot}7854d^2 \times f_s}{ptf_t} = \frac{1{\cdot}1781d^2}{pt}, \text{ since } f_s = \tfrac{3}{4}f_t \quad . \quad . \quad \mathbf{13}$$

Bearing Efficiency: $$\frac{2dtf_b}{ptf_t} = \frac{3d}{p}, \qquad \text{since } f_b = \tfrac{3}{2}f_t \quad . \quad . \quad \mathbf{14}$$

If failure occurs when the ideal joint is tested to destruction, then **12**, **13** and **14** should fail simultaneously, *i.e.*, **12** = **13** = **14**.

$\mathbf{12 = 13}$ $\quad p - d = 1{\cdot}1781d^2 \div t$, whence $p = 1{\cdot}571d^2 + d$. **14a**
$\mathbf{12 = 14}$ $\quad p - d = 3d$, whence $p = 4d$ . . . **14b**
$\mathbf{13 = 14}$ $\quad 1{\cdot}571d^2 = 3d$, whence $d = 1{\cdot}9''$ . . . **14c**

Equation **14c** gives an impossible diameter of rivet, so use the usual maximum of 1 in. The substitution of this value in **14a** and **14b** gives a pitch of 2·57 in. and 4 in. respectively; the former value is not permissible owing to the $3d$ rule.

The respective efficiencies for **12**, **13** and **14** are, with a 1-in. diameter rivet at 4 in. pitch, 75 per cent., 39·3 per cent. and 75 per cent.

The above shear efficiency is low and, obviously, can be increased by using the minimum pitch of $3d$. Adopting 1 in. diameter rivets at 3 in. pitch, the respective efficiencies are now $66\frac{2}{3}$ per cent., 52·4 per cent. and 100 per cent. By closing up the pitch from

4 in. to 3 in. the joint efficiency has risen from 39·3 per cent. up to 52·4 per cent.

Maximum Efficiency of a Double-riveted Lap Joint cannot be larger than 75 per cent., no matter what the diameter of the rivet and plate thickness may be. This is observable from equations **13** and **14**, which give $t = 0{\cdot}393d$; using this value and a pitch of $4d$, the efficiencies all run out at 75 per cent.

DOUBLE-RIVETED, DOUBLE-BUTT STRAP JOINT, FIG. 59.

Plate Efficiency: $$\frac{(p-d)tf_t}{ptf_t} = \frac{p-d}{p} \quad . \quad . \quad . \quad . \quad . \quad \mathbf{15}$$

Shear Efficiency: $$\frac{2 \times 0{\cdot}7854d^2f_{ds}}{ptf_t} = \frac{2{\cdot}3562d^2}{pt} \quad . \quad . \quad . \quad . \quad . \quad \mathbf{16}$$

There are two rivets each in double shear, and $f_{ds} = \frac{3}{2} f_t$.

Bearing Efficiency: $$\frac{2 \times dtf_b}{ptf_t} = \frac{3d}{p} \quad . \quad . \quad . \quad . \quad . \quad . \quad \mathbf{17}$$

Example.—Design a double-riveted, double-butt strap joint for $\frac{7}{8}$ in. thick main plates; no restriction is placed upon the rivet diameter.

Repeat the procedure of the previous example.

15 = **16**	$p - d = 2{\cdot}3562d^2 \div \frac{7}{8}$,	whence $p = 2{\cdot}6928d^2 + d$	**17**a
15 = **17**	$p - d = 3d$	whence $p = 4d$ . . .	**17**b
16 = **17**	$2{\cdot}3562d^2 \div \frac{7}{8} = 3d$	whence $d = 1{\cdot}114''$ . .	**17**c

Equation **17**c fixes the rivet diameter at $1\frac{1}{8}$ in., the nearest working size, and on substituting this value in **17**a and **17**b the pitch has the two values of 4·39 in. and 4·5 in. The working pitches are therefore $4\frac{3}{8}$ in. and $4\frac{1}{2}$ in.

Try $1\frac{1}{8}$-in. diameter rivets at $4\frac{3}{8}$-in. pitch. The efficiencies **15, 16** and **17** are 74·3 per cent., 77·9 per cent. and 77·1 per cent., while the same rivets at $4\frac{1}{2}$-in. pitch give the respective efficiencies of 75 per cent., 75·7 per cent. and 75 per cent. The latter pitch is to be preferred because, in addition to giving a slightly higher efficiency, it eliminates eighths of an inch from the pitch dimension.

Assuming that practice could make and use a rivet 1·114 in. in diameter, then the pitch obtained by substituting this value for d in **17**a and **17**b is, for both equations, 4·456 in.—again an impractical dimension. If this theoretical diameter and pitch of 1·114 in. and 4·456 in. be used in equations **15** to **17**, then the efficiencies all work out at 75 per cent. exactly. It is because the theoretical diameter is departed from in favour of a practical size of rivet that the apparent anomaly in the matter of the pitch arises.

Covers should be $\frac{5}{8}$ of $\frac{7}{8}$ in. each $= 0{\cdot}54$ in. For usual practice $\frac{1}{2}$-in. plates would be adopted. The B.S.S. (4/22) asks for 5 per cent. excess of cover area only, so the $\frac{1}{2}$-in. covers fully satisfy the specification.

Maximum Efficiency of a Double-riveted Butt Strap Joint with double covers can never rise above 75 per cent., and this occurs with a pitch of 4 diameters. Should the pitch be greater than 4 diameters the value of the bearing efficiency falls below 75 per cent., while that of the plate efficiency rises above 75 per cent. If the pitch be less

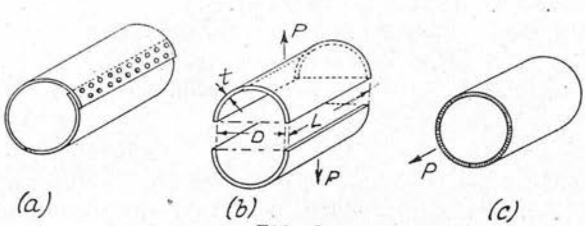


FIG 60

than 4 diameters the plate efficiency falls below 75 per cent., while the bearing efficiency rises above 75 per cent.

As a further example of a riveted joint the design of a riveted mild steel hydraulic main will be considered.

In Fig. 60 the resultant pressure tending to blow the pipe into two portions is $P = q \times D \times L$, where

q = Pressure in pounds per square inch.
D = Diameter of the thin shell or pipe in inches.
L = Length of the portion considered, in inches.

If S is the tensile stress in pounds per square inch of the metal, then the resistance offered by a seamless pipe would be S × area of metal, *i.e.*, $StL \times 2$, where t = pipe thickness in inches.

Hence $$2StL = qDL \quad \text{or} \quad S = \frac{qD}{2t} \quad . \quad . \quad . \quad . \quad . \quad 1$$

If one end of the cylindrical shell is closed the total pressure acting on it is $q \times$ area of end $= \frac{q\pi D^2}{4}$. This is resisted by the annular ring of metal shown hatched in diagram 60*c*. The area of this ring is approximately πDt, and its resistance is πDtS.

Hence $$\pi DtS = \frac{q\pi D^2}{4}, \quad \text{or} \quad S = \frac{qD}{4t} \quad . \quad . \quad . \quad . \quad 2$$

The stress of equation **2** is half that of equation **1**, so that the circumferential stress is half the longitudinal stress, and therefore circumferential joints need only be half the strength of the longitudinal joints.

The plates for the pipes are edge-planed with a bevel to facilitate caulking. They are then rolled or bent to cylindrical form and riveted longitudinally. The various small cylinders, each from 8 to 15 or 20 ft. long, are arranged so that every second cylinder is slightly larger in diameter than its immediate neighbours. This permits the smaller diameter pipes to be threaded into the larger and then riveted up circumferentially.

Circumferential joints are single-riveted lap joints, while the longitudinal ones are also lap joints, and are either single or double riveted as required. Butt joints are seldom used in the fabrication of mild steel hydraulic mains.

Numerical Example.—A hydraulic main 36-in. internal diameter is subjected to a maximum internal pressure of 200 lb. per square inch. Design the longitudinal joint. Take the permissible tensile stress on mild steel as 7 tons per square inch for this class of work.

Since a joint weakens a plate, the strength of the joint is the strength of the pipe. Let the efficiency of a double-riveted lap joint be E per cent. and T the required thickness of plate, then the effective thickness of the plating T is only $\frac{E}{100}$ of T. If it were possible to have a seamless pipe of the same diameter, then the thickness of this pipe would only require to be t, *i.e.*, t is less than T.

$$\therefore \qquad \frac{E}{100} \times T = t = \frac{qD}{2S}, \text{ or } T = \frac{100\,qD}{2ES}.$$

$S = 7 \times 2{,}240$ lb. per square inch.

$E =$ An efficiency of 70 per cent. to 75 per cent. for double-riveted lap joints.

$$\text{Hence} \qquad T = \frac{100 \times 200 \times 36}{2 \times 70 \times 7 \times 2{,}240} = 0{\cdot}328 \text{ inches.}$$

A $\frac{5}{16}$-in. plate is 0·3125 in., but as $\frac{1}{16}$ in. is often added to the thickness, so as to lengthen the life of the pipe in view of corrosion, the metal adopted would be of $\frac{3}{8}$ in. plating (0·375 in.).

After a tentative trial the riveting was fixed at $\frac{3}{4}$-in. diameter at $2\frac{1}{2}$-in. pitch. The efficiencies work out at :—

$$\text{Plate:} \qquad \frac{p-d}{p} = \frac{2\frac{1}{2} - \frac{3}{4}}{2\frac{1}{2}}, \qquad \text{or 70 per cent.}$$

$$\text{Shear:} \qquad \frac{1{\cdot}1781 d^2}{pt} = \frac{1{\cdot}1781 \times \frac{3}{4} \times \frac{3}{4}}{2\frac{1}{2} \times \frac{3}{8}}, \text{ or 70·7 per cent.}$$

$$\text{Bearing:} \qquad \frac{3d}{p} = \frac{3 \times \frac{3}{4}}{2\frac{1}{2}}, \qquad \text{or 90 per cent.}$$

The other dimensions adopted would be: edge of plate to rivet line = $1\frac{1}{4}$ in. (minimum $1\frac{1}{2}d$), rivet line to rivet line = $2\frac{1}{4}$ in. (minimum for diagonal pitch and tearing is $\frac{7}{8}$ of $p = 2{\cdot}16$), and rivet line to plate edge = $1\frac{1}{4}$ in.

Section weakened by One Rivet Hole.—Fig. 61 represents a very common type of joint. Bridge suspenders and tension diagonals were frequently made of flat bars, but this practice is not nearly so common now as formerly. It is an interesting example of how an "engineering trick" can save material, if the assumption as to stress distribution is correct.

Numerical Example.—A general drawing shows a flat bar 13″ wide × 1″ thick as being jointed or spliced at a certain point, but no information is given save that the section is to be weakened by only one rivet hole. Since neither the working stresses are given nor the number of rivets, the designer must fall back upon the fact that $f_{ds} = 2f_s = f_b = \frac{3}{2}f_t$. Design the connection.

Diameter of Rivet.—Unwin's rule, $d = 1{\cdot}4\sqrt{t}$ or $1{\cdot}2\sqrt{t}$, gives a diameter of 1·4 in. or 1·2 in.; the other rule, d not less than t, gives a 1-in. diameter rivet. Adopt the usual outsize of rivet, namely, 1 in. diameter.

Rivet Values.—Considering bearing alone. The sum of the thicknesses of the covers will be greater than the main plate thickness. Since the covers supply the reaction to the main plate both the latter and the former will have the same load to carry, and, therefore, if bearing failure occurs it will do so at the main plate.

The rivets are in double shear and bearing on a 1-in. thick plate.

Double shear value of a 1″ diameter rivet

$$= 0{\cdot}7854 \times \tfrac{3}{2}f_t = 1{\cdot}2\, f_t \text{ tons.}$$

Bearing value of a 1″ diameter rivet on 1″ Pl.

$$= 1'' \times 1'' \times \tfrac{3}{2}f_t = 1{\cdot}5\, f_t \text{ tons.}$$

Method of Failure 1.—All the rivets through one main plate will give way by double shear, since the rivet is weaker in double shear than in bearing.

Net sectional area of bar at A = (13″ − 1″)

$$\times\ 1'' \text{ tk.} = 12 \text{ sq. in.}$$

Permissible tensile load at A = 12 sq. in.

$$\times\ f_t \text{ tons/sq. in.} = 12\, f_t \text{ tons.}$$

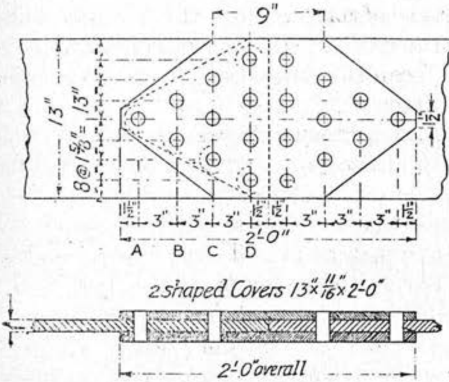
∴ Number of rivets required to develop this

$$\text{load} = 12f_t \div 1{\cdot}2\, f_t = 10.$$

The rivets are arranged as in diagram for the following reason.

Method of Failure 2.—If the main plate failed by tearing at B the left portion could not leave the cover plates until it had double-sheared the rivet at A as in Fig. 62. With a tear at C there are three rivets to double-shear, viz., one at A and two at B, before the main bar can travel to the left.

The total strength of the joint at any point on the main plate is thus the sum of the net tensile strength at that section plus the



MEMBER WEAKENED BY ONE RIVET HOLE

FIG. 61.

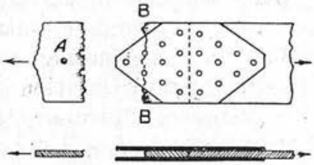


FIG 62

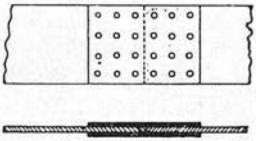


FIG 63

value of the rivets (either in double shear or bearing, whichever is the lesser) between the fracture and the end of the cover.

$$
\begin{array}{llll}
\text{Strength at A} & & & = 12f_t. \\
\quad ,, & \text{B} = & 11f_t + 1{\cdot}2f_t & = 12{\cdot}2f_t. \\
\quad ,, & \text{C} = & 10f_t + 3 \text{ rivets @ } 1{\cdot}2f_t & = 13{\cdot}6f_t. \\
\quad ,, & \text{D} = & 9f_t + 6 \text{ rivets @ } 1{\cdot}2f_t & = 16{\cdot}2f_t.
\end{array}
$$

Method of Failure 3.—Covers tearing straight across on the line of the four rivet holes at D.

The tensile value of the two covers should be together equal to that of the main plate at A.

$$
\begin{array}{ll}
\therefore & 2\,(13'' - 4'') \times t'' \times f_t = 12f_t \text{ (where } t = \text{cover thickness),} \\
& \text{whence} \qquad\qquad t = 0{\cdot}66''.
\end{array}
$$

It cannot be definitely stated that each cover takes half the pull on the main bar, although we assume this. To make allowance for any eccentricity the B.S.S. (4/22) asks that 5 per cent. more cover area be given than is actually necessary. The thickness t of each cover is thus increased to 0·69 in. Adopt $\frac{11}{16}$-in. cover (0·6875 in.).

Joint Efficiency.—Strength of plate not holed $= 13'' \times 1'' \times f_t$ tons per square inch $= 13f_t$, whence the efficiency $= 12f_t \div 13f_t$, *i.e.*, 92 per cent.

If three rows of four rivets per row had been adopted on each side of the joint as in Fig. 63, the efficiency would have been $9f_t \div 13f_t$, or approximately 70 per cent., failure taking place by tearing: thus by giving more rivets the joint may be made considerably weaker.

Shape of Cover.—The covers are tapered, but never encroach on the $1\frac{1}{2}d$ distance from the rivet centres, as indicated by the broken line circles round several of the rivets. The plate could have been closer cut after the manner of the dotted outline on the left-hand side, but this makes the joint look too sharp.

The cover is tapered or shaped because at the end it picks up one rivet value of strength, and should have therefore a width of plate

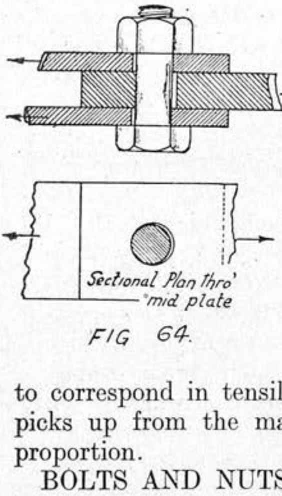


FIG 64.

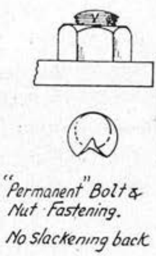


FIG. 65.

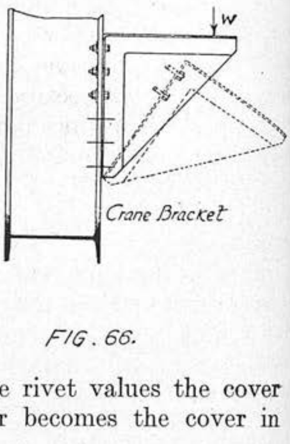


FIG. 66.

to correspond in tensile value. The more rivet values the cover picks up from the main plate the wider becomes the cover in proportion.

BOLTS AND NUTS are generally used in structural work as temporary fastenings to bind the material together during manufacture.

In the case of the ordinary manufactured mild steel bolt and nut (or black bolt as it is termed) the shank is $\frac{1}{16}$ in. less in diameter than the hole which it enters. As Fig. 64 plainly shows, the shank has only line contact throughout its length against the sides of the hole, instead of surface contact as with the machine-closed rivet. Because of this there will be more "give" in a structure when bolts are employed than when rivets are used. It will be deduced, therefore, that where the shank is subjected to shear and bearing the rivet is the more suitable of the two fastenings. Further, since the bolt does not completely fill the hole, it will have more room to deflect and the bending action of Fig. 53 will be intensified.

Nuts, when screwed up without any special precaution, are apt to work loose if the structure is exposed to any vibration. Where vibration is present the part of the bolt projecting beyond the nut is hammered out, and in that way the nut and bolt are locked together. This splaying out of the $\frac{1}{4}$-in. projection is achieved either by cold knobbling—*i.e.*, burring up with a hammer—or by cutting a longitudinal V-shaped chase or groove across the screw threads with a cold chisel. If the upper bolts of the crane bracket connection, Fig. 66, are so treated there will be no slackening back of the nuts. Fig. 65 shows the effect of the V groove.

In the connections of roof girders to columns and roof truss shoes to columns or to roof girders, bolts are often used, *i.e.*, in the connection of one portion of a structure to another, but not in the individual elements composing each portion.

Roof truss members are shop-riveted where possible and the truss sent to the site in as few parts as practicable. These parts are field-riveted or bolted at ground level on the site and the completed truss lifted up to its final position, either with an erection pole or crane, and the end connections bolted. These latter connections are high up in the air, and in all probability bolts would give as good a job as rivets because of the difficulty of satisfactory riveting. If there are only a few holes in each connection no scaffolding would be erected. The rivet forge would be at ground level and the rivets sent up in a bucket, pulled hand over hand by a rope, to the detriment of the rivet, which would be cooled. In cases of this kind bolts are permitted, in fact preferable, but subject to special provision as follows : Calculate the number of shop rivets required and increase their number by 20 per cent. if black bolts are to be used instead of rivets. B.S.S. (3/18).

Bright, fitted or turned bolts are machine-turned mild steel bolts and are made to give a tight fit at entry, the B.S.S. (2/9) specifying a maximum clearance of 0·005 in. In contradistinction to black bolts, they are always provided with washers. They are therefore much superior to black bolts, and are taken as being equivalent to shop rivets of the same diameter.

Fastenings under Tension.—Bolts should always be used where there is a possibility of longitudinal or axial tension, as in the upper bolts of Fig. 66. In this example the fastenings at the top have a force exerted upon them tending to pull off their heads. The resisting area is the area at the bottom of the thread of the bolt. It is by no means exceptional to have a pressure machine-closed rivet head fly off on cooling after being riveted up. To appreciate this point thoroughly consider the range of temperature which a rivet may have. From ordinary temperature up to a light cherry-red heat represents, say, a range of 1,500° F. A rod 2·973 in. long on heating through this range expands to 3 in., and conversely a 3-in. rod at light cherry-red contracts on cooling to 2·973 in.

The pull necessary to extend a rod 2·973 in. long up to 3 in. is found from the following :—

Young's modulus, $\quad E = \text{Stress} \div \text{Strain},$
whence $\quad \text{Stress} = E \times \text{Strain}.$

If $E = 13{,}000$ tons per square inch,

then
$$\text{Stress} = 13{,}000 \times \frac{3 - 2{\cdot}973}{2{\cdot}973}$$
$$= 118 \text{ tons per square inch.}$$

This is not a fair comparison perhaps, but nevertheless it illustrates in a striking manner that the rivet shank must be under high axial tension. The pulling inwards of bulging walls of unstable buildings is accomplished in the same manner, viz., by heating the tie rod and screwing up tight when the rod is expanded. On the rod cooling it contracts and exerts an inward pull on both walls.

Even in a bolted connection the bolt is not free from initial tensile stress. The tight screwing up of the rod tends to elongate the shank and stress must occur in proportion to the strain produced. There is, in addition to this, a slight torsional effect. This screwing up stress is more dangerous in small diameter than in large diameter bolts ; in fact a small bolt can be snapped in two by overtight screwing. By limiting the lever arm of the spanner to 15d a rough control is exercised upon the workman. See Chapter VIII. for the suggested permissible axial tensile stresses on rivets and bolts.

Rule for Area at the Bottom of the Thread.*—Convert the bolt diameter into eighths of an inch, then subtract one-eighth from this total. These two numbers when multiplied together and divided by 100 give a very close approximation to the required area.

Thus 1-in. diameter bolt, area at bottom of thread $= 8 \times 7 \div 100 = 0{\cdot}56$ sq. in. (correct area $= 0{\cdot}554$) ; $\frac{7}{8}$-in. diameter bolt, area at bottom of thread $= 7 \times 6 \div 100 = 0{\cdot}42$ sq. in. (correct area $= 0{\cdot}422$).

Foundation Bolts are usually larger in diameter than the ordinary fastenings, and are about 30 diameters long, see Chap III Vol II. They may have to take uplift, *i.e.*, they are in tension, or shear and bearing stresses, or a combination of these. Washers are always used with foundation bolts, as the holes have a clearance of $\frac{1}{8}$ in. or $\frac{1}{4}$ in. over the bolt diameter. The washer helps in spanning the hole and in distributing the load on to the nut. These bolts are in the majority of cases grouted in with cement, as also the nuts, or they are covered

* Rule by Mr. W. Stevenson on p. 384 (eighth edition), Goodman's *Mechanics Applied to Engineering*.

with concrete, so that there is little possibility of their becoming undone, intentionally or otherwise, the concrete serving as a lock nut.

Lock Nuts may be used instead of knobbling and chasing, but are seldom used in constructional work.

Tapered Washers are necessary where the outer member,

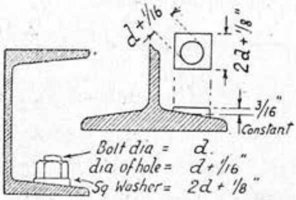


FIG 67.

penetrated by a bolt, has a splayed surface. Fig. 67 illustrates their use.

Shop and Field Rivets.—As is to be expected from what has been said, the rivet completed in the shop is a much superior rivet to that driven at the site. The B.S.S. (3/18) differentiates sharply between these two types of rivets by requiring that rivets if site-driven are to have their number increased by 15 per cent. over the number which would have been necessary had they been shop rivets.

REFERENCES

SPOONER. *Machine Design, Construction and Drawing.* (Longmans & Co.)

UNWIN, W. C. *Elements of Machine Design, Part I.* (Longmans & Co.)

GOODMAN, J. *Mechanics Applied to Engineering.* (Longmans & Co.)

British Engineering Standards Association. *Standard Specification for Dimensions of Rivets*, No. 275, 1927.

British Engineering Standards Association. *Standard Specification for Girder Bridges*, No. 153, *Parts III., IV. and V.*, 1923.

CHAPTER V

FLANGE PLATE SPLICES

Zigzag Tearing and Net Areas.—A student is apt to lay gre stress upon the fact that a flat bar with a central hole drilled in fails, under axial tension, at a higher load per net square inch than similar bar without a hole. This is due to the help given by t supposed non-active metal behind the hole, parallel to the directi of pull. Just prior to breaking, the metal tends to narrow at th point where the fracture occurs, and the surplus metal behind t hole restricts this contraction of the strained metal and th increases the ultimate strength. When there are several holes o of a member they are generally reeled, with perhaps only two rive out at any cross-section of the bar, as in Fig. 68. Now the stre

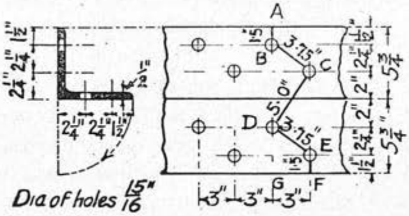


FIG. 68.

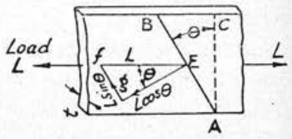


FIG. 69.

in the bar can hardly be expected to jump from side to side in ord to clear the holes without some loss of strength, so that the assum gain mentioned at the commencement of the paragraph can scarce be counted upon. Further, it brings in that debatable point as whether the elastic limit or the ultimate strength should be us when fixing the working stresses.

Fig. 69 is one which finds a place in nearly every text-book up strength of materials. The load L is axial on the flat bar, who width is AC and thickness is t. The tensile load per square inch

$$\text{AC} = \text{L} \div \text{AC} \times t \quad . \quad . \quad . \quad . \quad . \quad .$$

Resolving the load L normal to and along the diagonal plane A whose length = AC sec θ, then the shear along AB, per square inc

$$= \text{L} \sin\theta \div \text{AC} \sec\theta \times t = \text{L} \sin\theta \cos\theta \div \text{AC} \times t \quad .$$

Tensile stress normal to AB, per square inch

$$= \mathrm{L} \cos \theta \div \mathrm{AC} \sec \theta \times t = \mathrm{L} \cos^2\theta \div \mathrm{AC} \times t \quad . \quad . \quad \mathbf{3}$$

Value **3** can never be greater than that of **1**, since cos θ never exceeds unity, but plane AB, although it has a smaller normal tensile stress on it than AC, has the additional shearing stress.

Experimental results are rather erratic, but they all tend to show that a bar with reeled holes has a distinct penchant for tearing along the zigzag lines instead of straight across by the shortest route. That is, it tends to tear with the grain of the metal. When the diagonal line has a net tearing length (*i.e.*, between the holes and not centre to centre of holes) of about 30 per cent. in excess of the net straight across length, the tear occurs as readily one way as the other. If AC = 100, say, and AB = 130, then the amount to be allowed for AB by the B.S.S. is 0·8 of 130, viz., 104, thus meeting the experimental evidence.

The Canadian Society of Civil Engineers specifies that "There shall be deducted from each member as many rivets as there are gauge (rivet) lines, unless the distance centre to centre of rivets measured in the diagonal direction is 40 per cent. greater than their distance centre to centre of gauge lines."

The figure 40 per cent. is apparently higher than the B.S.S., but, be it noted, it is centre of rivets and not net lengths between edges of rivet holes.

Cooper specifies, "The rupture of a riveted tension member is to be considered as equally probable, either through a transverse line of rivet holes or through a zigzag line of rivet holes, where the net section does not exceed by 30 per cent. the net section along a transverse line."

The figures given for the pitches of Table 10 and for the areas of Table 11, Vol. III., to meet the requirements of the B.S.S. on zigzag tearing, were obtained by assuming the angles flattened out, as in Fig. 68, so as to give equivalent flat bar sections.

Length of Flange Plates.—The edges of flange plates (if so specified) are planed by machines, which are generally anything from 10 ft. to 30 ft., or even 40 ft. long. The machines usually, or should, have open ends, so that if a plate is longer than the machine it can be planed throughout its full length by pulling it along and resetting it. The plate is held in position by vertical jacks—screw or hydraulic—varying in number up to about a dozen, depending upon the length and make of the machine. These jacks have to be unloosened, the plate shifted by crane and trued up anew, and, finally, the jack pressure re-exerted. The time spent in the foregoing operations is much greater than the actual time spent in the machining.

Further, it was noted in Chapter I., under the heading of extra charged on plates, that long narrow plates could not be sheare with sides exactly parallel. The longer the plate the greater is th twist and, consequently, the amount of planing required is larger.

General British practice has thus limited the length of flang plates to about 30 ft., and splicing them if they exceed this length the extra material involved in the covers being more than paid fo by the time saved in handling the shorter plates.

In America the practice is to specify that the inner flange plate adjoining the flange angles, " shall run the full length of the girde continuously, without being spliced." Thus the $12'' \times \frac{1}{2}''$ plate o Fig. 70, if it occurred in a girder of 50 ft. length, would be unbroken whereas in British practice the plate might be spliced at 17 ft. fron each end, or, alternatively, say at the centre of the span, or som other suitable position, depending entirely upon the design and th designer.

No fixed or definite rule can be laid down regarding lengths c flange plates, and the designer settles each job upon its own merits Theoretically, try to do without splices, but, from a practical poin of view, remember cost. Since practice limits the length as statec it may be taken as axiomatic that a loss results from the use of long narrow plates.

Single Flange Plate Splice (Tension Flange).—Given a $12'' \times \frac{1}{2}$ flange plate, Fig. 70, and using $\frac{3}{4}$ in. diameter rivets, design the joint

By the B.S.S. (4/22) the plate shall be covered fully to develo the effective strength of the member. The effective strength = effective area (net in this case) $\times$ permissible stress. A transvers sectional area, as from **a** to **e**, will be assumed as being the effectiv area, and when the pitch has been fixed the question of the possibilit of diagonal tearing will be examined. It is assumed that no hel may be obtained from the main angles and web plate as these presumably, are working at their full capacity.

Looking at the small single line diagram, it will be seen tha failure may occur in two ways : firstly, by tearing of the cover plat at the joint, thus allowing the main plate to fly apart ; secondly through failure of the rivets either in bearing or in single shear a illustrated.

Rivet Values.

One $\frac{3}{4}$-in. diameter rivet in S.S. $= \frac{3}{4}f_t \times 0{\cdot}44 \quad = 0{\cdot}33f_t$ tons

One $\frac{3}{4}$-in. diameter rivet in bearing on $\frac{1}{2}$-in. plate $= \frac{3}{2}f_t \times \frac{3}{4}'' \times \frac{1}{2}'' = 0{\cdot}56f_t$ tons

The lesser of these two values is single shear and the rivets wi fail in S.S. in preference to bearing.

Plate Value, etc.

Net area of plate = (12″ − 2 holes @ $\frac{3}{4}$″) × $\frac{1}{2}$″ tk. = 5·25 sq. in.
Effective strength of plate = net area × f_t = 5·25 f_t tons.
Number of rivets required to develop plate
$$= 5{\cdot}25 f_t \div 0{\cdot}33 f_t = 16.$$
Net area of cover plate required = 5·25 sq. in.
+ 10 per cent. B.S.S. (4/22) = 5·78 sq. in.
Cover plate adopted 12″ × $\frac{9}{16}$″; (12″ − 2 @ $\frac{3}{4}$″)
× $\frac{9}{16}$″ = net area = 5·9 sq. in.

Both the rivets and the cover plate are at least as strong as the original plate.

The rivets are closely pitched in order to keep the cover plate

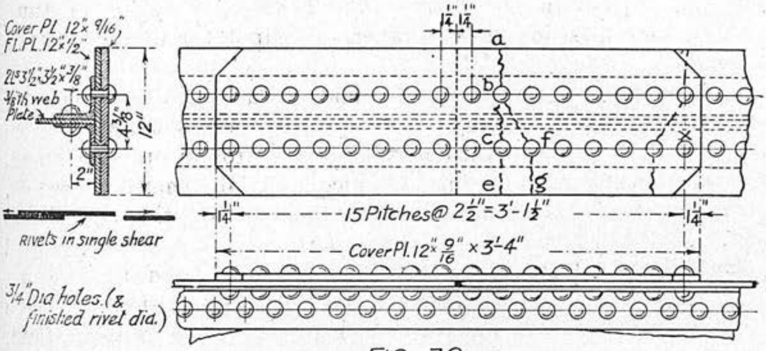


FIG. 70.

short; the pitch adopted is $2\frac{1}{2}$ in., which is larger than the minimum of $3d = 2\frac{1}{4}$ in.

Diagonal Tearing.—The 12″ × $\frac{9}{16}$″ cover plate may tear diagonally along the zigzag line **a**, **b**, **f**, **g** whose effective length in accordance with the B.S.S. (4/4) is

$$\left(\mathbf{ab} - \frac{d}{2}\right) + 0{\cdot}8\,(\mathbf{bf} - \mathbf{d}) + \left(\mathbf{fg} - \frac{d}{2}\right) =$$
$$3{\cdot}44'' + 3{\cdot}4'' + 3{\cdot}44'' = 10{\cdot}28''$$

Where **d** is the diameter of the rivet, viz. $\frac{3}{4}$ in.

Net transverse length **a e** is 12″ − 2 @ $\frac{3}{4}$″ = 10·5″.

The effective area of the cover plate against tearing = 10·28″ × $\frac{9}{16}$″ = 5·78 sq. ins., which was the exact amount required. The pitch, therefore, does not require to be increased because of diagonal tearing. The zigzag tearing, mentioned above, will occur right over the joint of the flange plate, but for clearness the line **a b f g** has been shown to the right of the joint.

Should the $12'' \times \frac{1}{2}''$ main plate tear diagonally it will do so nea the end of the cover plate at **X**, where it is not fully reinforced b the cover plate. This is assuming, of course, that the rivets ar closely pitched only in the length of the cover, and that whe outwith the cover the rivets are spaced far enough apart to ensur a transverse tearing taking place. The effective area of the mai plate is the diagonal effective length × thickness

$$= 10{\cdot}28'' \times \tfrac{1}{2}'' \text{ net} = 5{\cdot}14 \text{ sq. in.}$$

But before the main plate can fly apart it must shear the rivet at **X**

(This will be single shear since, according to our original assump tion, the angles are virtually non-existent.)

The total value of the main plate and rivet at **X** is thus $5{\cdot}14f_t$ + $0{\cdot}33f_t = 5{\cdot}47f_t$, *i.e.*, greater than the main plate's standard value o $5{\cdot}25f_t$ tons.

Since the rivets in the main angles of the flange reel with thos in the flange plate, the net area of the tension flange angles woul require to be checked for diagonal tearing due to close pitching o the rivets in the length of the splice. However, if possible, a flang splice is never placed near the point of maximum bending momen but always some distance away from this point. There is, therefor the probability of an excess of flange area at the splice, which permit the designer to take away a small amount of the flange angle are owing to the close pitching of the rivets.

The rivet pitch can be opened out after passing the flange plat splice, so that at the point of maximum bending moment (and therefore, minimum shear) the pitch is wide, and the net transvers area, and not the zigzag area, will be the minimum.

Single Flange Plate Splice (Compression Flange).—Main sectio as for tension flange.

The practice of several firms used to demand that the splice of compression menber should be designed as if that member were i tension and not in compression. From this point of view the abov joint would do for both tension and compression flanges, and it use in the compression flange would certainly fulfil the require ments of the B.S.S., as the under-noted calculations show.

Plate Value, etc.

Gross area of plate $12'' \times \frac{1}{2}''$ $= 6$ sq. in.

Effective strength of main plate = gross area × permissible compressive stress $f_c = 6f_c$ tons.

Number of rivets required to develop plate $= 6f_c \div 0{\cdot}33f_t = 18f_c \div f_t$.

Gross area of cover plate required = 6 sq. in. + 10 per cent. = 6·6 sq. in.

Cover plate adopted $12'' \times \frac{9}{16}''$, gross area of which = 6·75 sq. in

The permissible f_c will be known from the design of the main girder flanges, and when substituted in the above equations will give the required number of rivets.

Checking the Tensile Joint used as a Compressive Joint.—From the formula of the B.S.S. (3/18) the permissible stress in compression $f_c = f_t\left(1 - 0{\cdot}01\,\frac{l}{b}\right)$. The term $\frac{l}{b}$ may vary from 1 to 40 by the B.S.S. (4/11), and hence f_c may vary between $0{\cdot}99f_t$ and $0{\cdot}6f_t$. The term $\frac{l}{b}$ usually lies between 15 and 40, and, taking as a common and much used value of, say, 25, the usual permissible f_c works out at $\frac{3}{4}f_t$.

Hence the number of rivets required $= 18f_c \div f_t = 13{\cdot}5$, say 14 rivets.

The net or tension method gave 16 rivets, and is therefore well on the safe side.

The cover plate of $\frac{9}{16}$ in. found by the tensile method is also sufficient from the point of view of compression. Diagonal tearing does not require to be considered in a compression member [B.S.S. (4/4)], and so the design is simplified accordingly. In the compression member the rivet pitch should, therefore, be as near the minimum as possible in view of the strut action of the individual plates, and also to shorten the cover plate for economy.

Alternative Single Flange Plate Splice (Tension).—Fig. 71 is an alternative design to that of the previous figure.

The distance between the toe of the flange angle and the edge of the flange plate is $2\frac{5}{16}$ in. The rivets are $\frac{3}{4}$ in. diameter, so that if cover strips are used on the inside face of the flange plate the minimum width requires to be $3d$, *i.e.*, $2\frac{1}{4}$ in. By using a flat or plate of $2\frac{1}{4}$ in. width a clearance of $\frac{1}{16}$ in. is obtained at the toe of the main angles, and any irregularities of the angles will be cleared. Appearance fixed the thickness of these strips at $\frac{3}{8}$ in. to agree with the flange angle, and a flush surface is thus presented to the rivet die. This type of detail could not have been used if the flange plate had been narrower than 12 in.

The rivet pitch adopted is 6 in. (*i.e.*, 3 in. alternate pitch), because there are now four rivet lines along the plate offering a greater possibility for diagonal tearing than in the previous case of two rivet lines. With this width of pitch there is only transverse tearing of the flange angles. It is assumed that the girder shear calculations permit of a 6-in. pitch in the angles.

Covers tearing at the weakest section, *i.e.*, over the joint. Main cover $\frac{1}{2}$ in. thick.

Net transverse area of cover and two stripes

$$= (12'' - 2 @ \tfrac{3}{4})\,\tfrac{1}{2}'' \text{ plus } 2 @ 2\tfrac{1}{4}'' \times \tfrac{3}{8}'' = 6{\cdot}94 \text{ sq. in.}$$

Diagonal tearing of outer cover **a b c e f g**

$$= 10{\cdot}32'' \times \tfrac{1}{2}'' \quad = 5{\cdot}16 \text{ sq. in. net.}$$

Plus transverse tearing of strips

at **a** and **g** $= 2\,(2\tfrac{1}{4}'' - \tfrac{3}{4}'')\,\tfrac{3}{8}'' = \underline{1{\cdot}12}$ sq. in. net.

Total $= 6{\cdot}28$ sq. in.

This more than satisfies the required cover area of 5·78 sq. in. of the previous calculations.

Assuming the angles to be non-existent, the rivets connecting the inner cover strips, flange plate, and outer main cover are in double

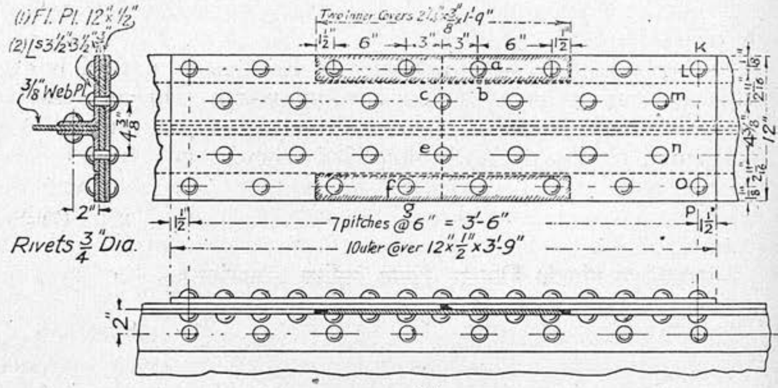


FIG. 71.

shear ($0{\cdot}66f_t$ per rivet) or bearing on the mid $\frac{1}{2}$-in. flange plate ($0{\cdot}56f_t$ per rivet).

Rivet Strength.—Ten rivets in S.S. + 4 in bearing $= 10 \times 0{\cdot}33f_t + 4 \times 0{\cdot}56f_t = 5{\cdot}54f_t$ tons, which is satisfactory, the effective main plate strength being $5{\cdot}25f_t$ tons.

No help is given by the two rivets on the line of the joint.

The rivets connecting the narrow strips to the flange plate are in double shear, made up of one single shear between the inner strip and flange plate, plus another single shear between flange plate and the outer cover. The inner strips are, therefore, attached by rivets which, so far as the individual strips themselves are concerned, are in single shear. The rivets connecting the strips to the flange plate should just develop the strength of each strip on each side of the joint.

Net area of one strip $= (2\frac{1}{4}'' - \frac{3}{4}'') \times \frac{3}{8}'' \qquad = 0{\cdot}5625$ sq. in.

$\therefore$ Number of rivets required to develop the

$$\text{strip} = 0{\cdot}56f_t \div 0{\cdot}33f_t \text{ (\textit{i.e.}, S.S. value)} = 2$$

Occasionally, however, the strips are carried the full length of the outer cover for appearance, but no extra strength is gained thereby as the plates would fail by tearing, while the connecting rivets would remain intact. Theoretically, both rivets and plates should have the same ultimate strength.

Main Flange Plate Tearing Diagonally.—This would occur at the weakest part, *k l m n o p*. The right portion of the flange plate would then travel towards the right hand by slipping out from under the outer cover and leaving it undisturbed. If the tear occurs nearer the joint there will always be some rivets to shear before the flange plate frees itself from the outer cover.

Effective diagonal area of flange plate

$$= \text{diagonal length} \times \text{thickness}$$
$$= 10{\cdot}32'' \times \tfrac{1}{2}'' = 5{\cdot}16 \text{ sq. in. net,}$$

which is slightly less than the net transverse area of 5·25 sq. in.

Spacing the end four rivets wider apart longitudinally than the others in the cover plate would tend to prevent diagonal or zigzag tearing of the flange plate, provided, of course, that the rivet pitch outwith the joint is wider than that within the spliced portion. On the other hand, the rivets at the ends of the covers are more highly stressed than the ones nearer the joint, and for this reason the rivets, theoretically, should be clustered near the end of the cover. By Hooke's law, strain or elongation is proportional to the stress or force, and therefore at *l.o.* in the main plate there will be maximum stretch since the load is at its maximum value. After *l.o.* has been passed the stress in the main plate has been lowered by two rivet values, which have been taken from the main plate and transferred into the cover. Similarly, after *m.n.* the stress in the main plate is further reduced by other two rivet values. The amount of stretch between each pair of rivets in a longitudinal direction of the main plate decreases towards the centre of the splice. That piece of the main plate which stretches the most will therefore throw a greater load on to its connecting rivets, and this occurs at *l.o.*

Practice disregards this anomaly by making all the pitches the same throughout the length of the cover. In this particular case the previous type of splice with no strips is to be preferred as it is simpler and requires less metal.

Special Case of Single Flange Plate Splice.—In Fig. 72 the plate which is being covered is one of the inner plates of the flange and not the outer plate. Notwithstanding the fact that other plates intervene between the broken plate and its cover, the design is carried through as if they were in direct contact. The method of procedure is, therefore, exactly similar to that employed in the calculations of the joint illustrated in Fig 70. Cover plate area = flange plate area plus 10 per cent., while there should be sufficient rivets in the cover plate, on each side of the joint, to develop in single shear the strength of the broken flange plate.

The intervening plates are not packers within the meaning of paragraph 4/28 of the B.S.S. The various flange plates are riveted together throughout their full length and may be assumed to form a solid mass of steel, Fig. 73. Into this "monolithic" mass an incision is made at the joint, and to cover this incision, gap, or

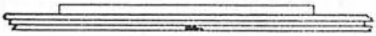


FIG. 72.

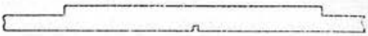


FIG. 73.

joint the cover plate is placed in direct contact with the outer flange plate.

It is worthy of note that the American Railway Engineering Association terms this an indirect splice, since the splice plate is not in direct contact with the parts which it connects. This specification demands that "rivets should be used on each side of the joint in excess of the number theoretically required to the extent of one-third of the number for each intervening plate." This is undoubtedly excessive in this case, as can be proved by following the shears from the angle surface outwards to the cover plate. However, if the grip of the rivets becomes greater than $4d$ the cover plate should be lengthened so as to include the necessary additional rivets, as previously specified, viz., 1 per cent. increase for each additional $\frac{1}{16}$ in. grip over four rivet diameters in length.

Grouped Flange Plate Splice.—In Fig. 74 there are three flange plates all broken near one another and all are covered by one continuous cover plate. Reading from the flange angles outwards, the plates are $20'' \times \frac{5}{8}''$, $20'' \times \frac{5}{8}''$, and the outermost plate $20'' \times \frac{1}{2}''$.

The rivet diameter was fixed at $\frac{15}{16}$ in. from the main calculations, and the values per rivet are $\text{S.S.} = 0{\cdot}52f_t$, $\text{D.S.} = 1{\cdot}04f_t$ and bearing on a $\frac{3}{8}$-in. plate $= 0{\cdot}52f_t$.

Since all the plates employed are thicker than $\frac{3}{8}$ in. rivet failure will occur in single shear, the least of the rivet values. This mode of failure is illustrated in Fig. 75.

A 3-in. reeled pitch, *i.e.*, a 6-in. pitch, will be used for the flange plate splice for the following reason. From Table 11 (Vol. III.), sectional areas of $6'' \times 6''$ angles with double rivet lines, it is found that with $\frac{15}{16}$-in. diameter rivets the zigzag tearing area is not less than the transverse tearing area, and so the splice rivet holes do not weaken the main angles in any way. This assumes, of course, that a closer reeled pitch than 3 in. is not required, which assumption is probably correct, as a simply supported girder will never require a splice of this type near the abutments where the shear is high and, therefore, the rivet pitch close.

Net transverse length *abdegh*

$$= 20'' - 4 \text{ @ } \tfrac{15}{16}'' \qquad = 16{\cdot}25''$$

Zigzag path *abcdefgh*

$$= 2(1{\cdot}5'' - \tfrac{15}{32}'') + 2[0{\cdot}8(4{\cdot}8'' - \tfrac{15}{16}'')] + 2[0{\cdot}8(3{\cdot}7'' - \tfrac{15}{16}'')] + (5'' - \tfrac{15}{16}'') \qquad = 16{\cdot}72''$$

Zigzag path *abcfgh*

$$= 2(1{\cdot}5'' - \tfrac{15}{32}'') + 2[0{\cdot}8(4{\cdot}8'' - \tfrac{15}{16}'')] + (9{\cdot}5'' - \tfrac{15}{16}'') = 16{\cdot}8''$$

Should the cover plate fail by tearing at points W, X or Y, it will do so through a transverse section on the line of the four adjacent

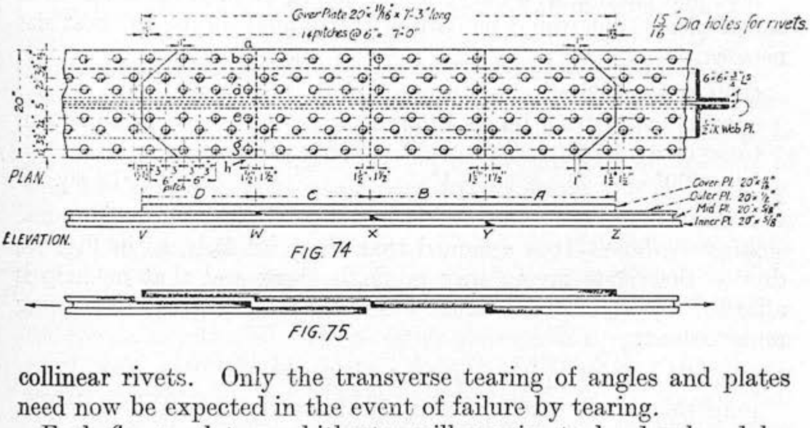


FIG. 74

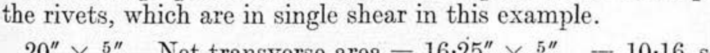


FIG. 75

collinear rivets. Only the transverse tearing of angles and plates need now be expected in the event of failure by tearing.

Each flange plate, as hitherto, will require to be developed by the rivets, which are in single shear in this example.

$20'' \times \tfrac{5}{8}''$.—Net transverse area $= 16{\cdot}25'' \times \tfrac{5}{8}'' \quad = 10{\cdot}16$ sq. in.
$20'' \times \tfrac{1}{2}''$.—Net transverse area $= 16{\cdot}25'' \times \tfrac{1}{2}'' \quad = 8{\cdot}13$ sq. in.
Rivets required for $20'' \times \tfrac{5}{8}''$ pl. $= 10{\cdot}16 f_t \div 0{\cdot}52 f_t = 20$ rivets
Rivets required for $20'' \times \tfrac{1}{2}''$ pl. $= 8{\cdot}13 f_t \div 0{\cdot}52 f_t = 16$ rivets

The unbroken or original strength of the flange plates

$$= 10{\cdot}16 f_t + 10{\cdot}16 f_t + 8{\cdot}13 f_t = 28{\cdot}45 f_t.$$

Cover Plate.—One method of joint failure is for the main plates and cover plates to tear straight through, and this would naturally occur at a point where one of the flange plates has its continuity broken by the joint. The strength in tension of the joint should everywhere be at least equal to the tensile strength of the original two $\frac{5}{8}$-in. flange plates plus one $\frac{1}{2}$-in. flange plate.

Thus, tearing at W,

2 unbroken $20'' \times \frac{5}{8}''$ pls. + cover pl. = 2 pls. $20'' \times \frac{5}{8}''$ + 1 pl. $20'' \times \frac{1}{2}''$ 1

Hence the cover pl. at W = at least the $20'' \times \frac{1}{2}''$ pl. + the B.S.S. requirement of 10 per cent.

Tearing at X

1 unbroken $20'' \times \frac{5}{8}''$ pl. + 1 unbroken $20'' \times \frac{1}{2}''$ + cover pl. = 2 pls. $20'' \times \frac{5}{8}''$ + 1 pl. $20'' \times \frac{1}{2}''$ 2

The cover pl. at X must at least = $20'' \times \frac{5}{8}''$ pl. + the B.S.S. requirement of 10 per cent.

By similar reasoning the cover pl. at Y should develop the $20'' \times \frac{5}{8}''$ pl. + 10 per cent. 3

The plate acting as a cover would not be tapered in thickness to suit **1** and **2**, but would be carried throughout at the thickest size necessary.

Cover area required = net area of $\frac{5}{8}''$ pl. + 10 per cent. = 10·16 sq. in. + 10 per cent. = 11·18 sq. in.

Cover area adopted = $20'' \times \frac{11}{16}''$, net area of which = $(20'' - 4 \text{ @ } \frac{15}{16}'') \times \frac{11}{16}''$ = 11·18 sq. in.

the transverse tearing area being less than the zigzag tearing area.

Rivet Failure.—It is assumed that the joint fails, as in Fig. 75, due to the rivets giving way in single shear and that no help is afforded by the main angles which may be regarded as being non-existent.

If joint W of the $\frac{1}{2}$-in. plate were the only joint then the D group of rivets = C group of rivets in S.S. *i.e.*, equal to the strength of the $\frac{1}{2}$-in. plate = 16 rivets in S.S. . 4

as previously explained for the single flange plate splice.

Similarly if X were the only joint, then group C in S.S. = group B in S.S. = strength of $20'' \times \frac{5}{8}''$ pl. = 20 rivets in S.S. . 5

And the joint at Y would have group B in S.S. = group A in S.S. = strength of $20'' \times \frac{5}{8}''$ pl. = 20 rivets in S.S. . 6

Summarising the above results and taking the greater where two values occur, groups D, C, B and A should have 16, 20, 20 and 20 rivets respectively.

The joint could fail owing to all the rivets giving way simultaneously in single shear as indicated in Fig. 75, *i.e.*, rivet groups D, C and B.

Value $= 16 + 20 + 20$ @ $0{\cdot}52 f_t$ per rivet in S.S. $= 29{\cdot}12 f_t$ tons.
The original strength of the flange plates was $=$
$(2$ @ $10{\cdot}16 + 8{\cdot}13) f_t$ $= 28{\cdot}45 f_t$ tons.

The joint is therefore as strong as the original plates, both as regards tearing and rivet failure.

Practice very often makes the rivets in groups A and D of sufficient number to develop in single shear the net tensile strength of the cover, *i.e.*, 10 per cent. in excess of the strength of the thickest plate covered **7**

A chain which can carry a load of 110 tons when fastened by an end hook which can only carry 100 tons is only useful for a 100-ton load. The same argument applies to the cover and its end rivets. The B.S.S. does not specify the extra rivets, but only the extra area.

Each group of rivets in the cover plate should develop the broken plate underneath it by the single shear (or bearing) value of the rivets. Groups A and B were given 22 rivets each, while groups C and D were given 20. It was aimed to give each group 20 rivets, *i.e.*, fully developing the $\frac{5}{8}$-in. plates, and making group A equal to group D as a compromise to **7**. The riveting did not permit of this, so the extra rivets were given at the heavy end of the joint. The cover plate is cut away at the ends as shown in the plan of Fig. 74, instead of straight across as with Fig. 71. This was done not only for appearance, but also for the following reasons :—

(1) The cover plate area should be increased from the end towards the centre in proportion as the cover plate picks up stress from the main plate through the rivets. At the end there are two rivets, therefore a narrow plate ; at the second row of rivets there is a total of $2 + 2$ rivets acting ; while at the third row there are now 8 rivets acting, and therefore a wider or stronger plate. This gives a more evenly applied load to the cover plate than the method of Fig. 71.

(2) It permits of the rivet die getting down to the flange plate rivet heads without fouling the edge or end of the cover plate.

(3) The end edge of the cover plate being closely adjacent to a rivet is held firmly to the main plate and no interstice is left for corrosion.

The dotted lines indicate how the cover ends were " struck " out on the drawing.

The above joint presents no difficulty, yet innumerable mistakes are made in its use. Cases are known where the cover plates on the drawings extended from V to X only, instead of from V to Z. For example, a draughtsman thought to improve upon the designer's joint and cut the cover length from VZ down to VX on the working drawings. The mistake was happily discovered in the yard just prior to shipment, and the rivets were cut out and the joint made good. The cost was rather large, as erection was delayed at the site.

Fig. 76 is a modified form of the foregoing flange splice.

Freight charges are made on either the weight or the overall cubic capacity of the material shipped, whichever is more favourable to the shipowner. A girder which is being sent by sea in small lengths would never be given the joint of Fig. 74 with its projecting plates, because the unriveted and comparatively flexible flange plates would get badly distorted. For these two reasons projecting ends of flange plates are never permitted when the girder is being sent abroad in suitable sizes.

The inner plate is jointed at the centre line of the detail marked C.L., and the other flange plates are stepped back from this line so

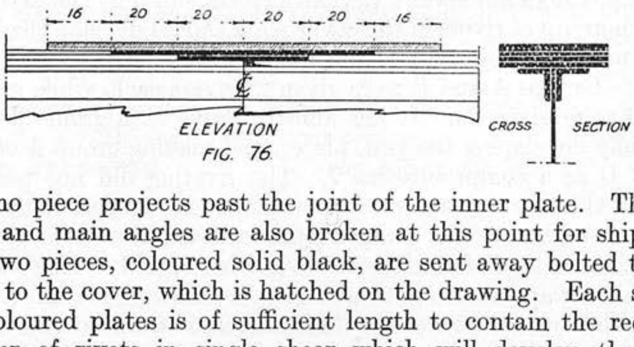


FIG. 76.

that no piece projects past the joint of the inner plate. The web plate and main angles are also broken at this point for shipment. The two pieces, coloured solid black, are sent away bolted temporarily to the cover, which is hatched on the drawing. Each step of the coloured plates is of sufficient length to contain the requisite number of rivets in single shear which will develop the plate immediately underneath it. On arrival at site the centre pieces are dropped into position and the whole riveted up.

If the black plates and hatched cover remain riveted to the right-hand portion of the girder, the left-hand portion can travel towards the left on single shearing the three groups of 16, 20 and 20 rivets on the left of the C.L. The main angles and web are not taken into account in providing help to the flange plates. The number of rivets in the end of the hatched cover theoretically only require to be 16 in order to develop the $\frac{1}{2}$ in. outer flange plate : see remarks above regarding the developing of the tensile strength of the cover by means of the end rivet groups.

The cover plate thickness is that of the thickest plate ($\frac{5}{8}$ in.) plus 10 per cent., *i.e.*, $\frac{11}{16}''$. This cover plate is approximately one and a half times the length of that used in the previous flange plate splice.

CHAPTER VI

SPLICES FOR ANGLES, JOISTS AND CHANNELS

ANGLES, joists, channels and tees can act individually, either as tension or, because of their rigidity, as compression members.

Where a splice occurs in any of these sections, when acting as tension members, the stress is abstracted from the section by the rivets on one side of the joint and transferred into the covers, is then carried in these covers across the joint, and ultimately finds its way back into the other portion of the section through the remaining group of rivets.

When the section carries an axial compressive load the theoretical function of the covers is to prevent lateral or side slip of the butting ends of the joint relatively to each other. The butting ends of compression members are machined to flat surfaces in order that they may bear evenly against each other, and thus the load is transferred from one part of the member to the other by direct contact of the machined ends. However, this perfect bearing of surfaces can only be attained by exercising elaborate care in workmanship, so that it may be questioned if the ideal case is ever realised in practice.

Probably a more important point than the foregoing is the eccentricity of the load on the section. There is generally, however unintentional, an eccentricity of loading which causes bending with consequent additional fibre stresses. Eccentricity of loading is more dangerous with a compression member than with a tension member, because in the former case an increase in the load is accompanied by an increased buckling or displacement of the centre line of the section away from the load line, while in the latter case the centre line exhibits a greater tendency to coincide with the load line. Fig. 136, of Chapter VIII., shows the load P applied longitudinally to an angle through the rivets on the rivet line. The eccentricity of the line of action of P from the centre of gravity line of the angle is e, and the resulting bending moment is Pe. This "unintentional" eccentricity is small, and its effects are always neglected in the design calculations. If the eccentricity is larger than that due to the rivet lines then special provision should be made in the covers and the rivets of the splice to carry both the direct load and the bending stresses created by the eccentricity, *i.e.*, if the load and not th member is to be developed by the covers and their rivets. However complete safety can be obtained by making the splice covers and rivets of both tension and compression members develop the ful net tensile strength of the section, even though this strength i much larger than the actual load carried. Therefore, no matte

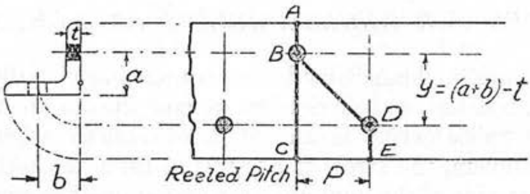
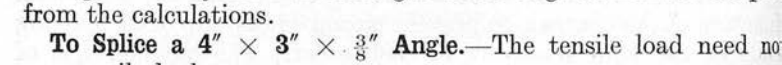


Illustration from Table 10, Vol. III.

whether machined ends butt or whether the load is axial or not, th strength of the joint is the strength of the original member adopted from the calculations.

To Splice a 4″ × 3″ × $\frac{3}{8}$″ Angle.—The tensile load need no necessarily be known.

As mentioned at the commencement of Chapter V., the variou possibilities of zigzag tearing are not fully known, and as an indica



Splice of a 4"×3"×3/8" Angle *Rivet Holes 3/4 Dia.*
Fig. 77

tion of the different requirements compare the following specificatio with the B.S.S. (4/4): "In calculating the net area of any tensio member the greatest number of rivets that can be cut by a trans verse plane perpendicular to the axis of the member, or come within 1″ (others state $1\frac{1}{2}$″ to 4″) of the plane, is to be deducted from the gross transverse area." By this specification, using the 3 in. pitch or $1\frac{1}{2}$ in. reeled pitch of Fig. 77, the net area of the angle to be developed is the net transverse area of 2·48 sq. in. gross — one hole $\frac{3}{8}'' \times \frac{3}{4}'' = 2{\cdot}20$ sq. in. net. By the B.S.S. (4/4) only straight through or transverse tearing would occur if the reeled pitch were increased to 4·13 in. See Table 10, Vol. III, "Sectional Areas of Angles with Single Rivet Lines."

The diagonal tearing of an angle is more easily considered if the angle is developed out into the equivalent flat. Thus in the key diagram to Table 10, Vol. III, but also given herewith, the $4'' \times 3'' \times \frac{3}{8}''$ angle would be equivalent to a rectangle $6\frac{5}{8}''$ wide (not $7''$) $\times \frac{3}{8}''$ thick. The two rivet lines would still be at the same distances from the edges, but would be $\frac{3}{8}$ in. nearer to each other. AB (of Table 10) $= 1\frac{3}{4}''$, $y = 2\frac{1}{4}'' + 1\frac{3}{4}'' - \frac{3}{8}'' = 3\frac{5}{8}''$ and $\text{DE} = 1\frac{1}{4}''$. With $\text{P} = 1\frac{1}{2}''$ the gross length of path $\text{ABDE} = 1\frac{3}{4}'' + 3{\cdot}923'' + 1\frac{1}{4}''$, while the net length in accordance with the B.S.S. is $(1\frac{3}{4}'' - \frac{3}{8}'') + (3{\cdot}923'' - \frac{3}{4}'')\,\frac{4}{5} + (1\frac{1}{4}'' - \frac{3}{8}'') = 4{\cdot}79''$; thus giving a net zigzag area of $4{\cdot}79'' \times \frac{3}{8}''$ thick $= 1{\cdot}80$ sq. in. net.

Gross cross-sectional area of angle	$= (4'' + 3'' - \frac{3}{8}'') \times \frac{3}{8}''$	= sq. in.	2·48
Net transverse area of angle	$= 2{\cdot}48 - 1$ hole $\frac{3}{4}'' \times \frac{3}{8}'' =$	,,	2·20
Net diagonal area of angle	= as before	= ,,	1·80
Tensile strength of angle	= net area $\times f_t$	= tons	$1{\cdot}80f_t$
Value of 1 rivet in D.S.	$= f_{ds} \times$ area $= 1{\cdot}5f_t \times 0{\cdot}44$	= ,,	$0{\cdot}66f_t$
Value of 1 rivet in $\frac{3}{8}''$ B	$= f_b \times$ area $= 1{\cdot}5f_t \times \frac{3}{8}'' \times \frac{3}{4}''$	= ,,	$0{\cdot}42f_t$
Number of rivets to develop angle	$= 1{\cdot}80f_t \div 0{\cdot}42f_t$	=	5
United width of angle legs	$= 4'' + 3''$	= in.	7
∴ Number of rivets in $3''$ leg	$= \frac{3}{7}$ of 5	=	2
∴ Number of rivets in $4''$ leg	$= \frac{4}{7}$ of 5	=	3

Covers.—Total net area req.	$= 1{\cdot}80 + 5$ per cent.: B.S.S. (4/22)	= sq. in.	1·89
Amount to $3''$ leg	$= \frac{3}{7}$ of 1·89	= ,,	0·81
Amount to $4''$ leg	$= \frac{4}{7}$ of 1·89	= ,,	1·08

Given :—$3''$ leg. 1 flat $3'' \times \frac{1}{4}''$ + 1 flat $2'' \times \frac{5}{16}''$
Gross area = 1·37 sq. in. total. Total net area = ,, 0·95
$4''$ leg. 1 flat $4'' \times \frac{1}{4}''$ + 1 flat $3'' \times \frac{5}{16}''$
Gross area = 1·94 sq. in. total. Total net area = ,, 1·52

It is desirable that the elongation of every cover should be the same, otherwise a larger load will be given to the more resisting plate with a consequent eccentricity of loading at the joint. For this reason, subject to practical sizes, the covers have been given the same area on each side of any one leg, and the total metal in the covers is proportional to the amount of metal in the leg which they cover.

Checking the Joint.

1. Rivets on one side of the joint failing by crushing, 5 rivets @ $0{\cdot}42f_t$; *i.e.*, 3 in 4″ leg and 2 in 3″ leg	= tons	$2{\cdot}1f_t$
2. Main angle failing by zigzag tearing. Strength was	= ,,	$1{\cdot}80f_t$
3. The covers on the 4″ leg tearing at A and those on the 3″ leg at B. Strength = net area × $f_t = 1{\cdot}52f_t + 0{\cdot}95f_t$	= ,,	$2{\cdot}47f_t$
Efficiency in tension $= 1{\cdot}80f_t \div 2{\cdot}48f_t = 0{\cdot}73$ or		73%

Rivet Pitch.—If the design is governed by the B.S.S., then any increase of the rivet pitch will give a corresponding increase of angle strength due to the longer diagonal path for tearing. The maximum pitch, however, cannot exceed $16t = 4''$ by the B.S.S. (4/26).

The 2-in. wide cover should be at least $2\frac{1}{4}$ in. wide, so as to agree with the rule "that the least distance from the centre of a rivet hole to the edge of the plate should be $1\frac{1}{2}d$." An attempt could be made to fulfil this condition by adopting a cover wider than 2 in., and grinding down one edge so as to fit into the root fillet of the angle, or, alternatively, by using an interior $\frac{5}{16}$-in. thick bent plate to cover both the 4-in. and the 3-in. legs of the original section.

The rivet shown dotted is a stitching rivet holding the covers close against the main angles, but it adds nothing to the strength of the joint. It is required because of the rule $16t = 16 \times \frac{1}{4}'' = 4''$ maximum pitch for tension members and $12t = 3''$ for compression members. The angles and covers are assembled, bolted together, and the hole then drilled through the covers and main angles.

When the $4'' \times 3'' \times \frac{3}{8}''$ Angle is a Strut.—The above joint is equally applicable when the angle acts as a strut. In fact, it would be overstrong if the load were axially applied. The load, however, is always applied to the angle by the rivets on the rivet line, and, therefore, the angle is non-axially loaded. Thus there is a bending moment, already spoken of as "unintentional," existing in both the tension and compression splices.

In the case of a compression member the thrust coming on the edge of a rivet-hole is transferred across that hole by the rivet shank, which is assumed to fill the hole completely. This does not apply to a tension member where the load is a pull. Net areas are, therefore, used in tension members, but gross areas for compression members.

Effective strength, B.S.S. (4/22)

$$= \text{effective area} \times \text{permissible stress,}$$
$$= \text{gross area} \times f_c \qquad = 2{\cdot}48 f_c$$

Now f_c the working compressive stress is generally considerably less than f_t the corresponding working tensile stress. The longer the angle strut the smaller is the working stress, but assuming an $\frac{l}{r}$ about 100, *i.e.*, the angle is in the region of 5 ft. long, the working compressive stress f_c will be about $0{\cdot}5f_t$, depending upon end conditions and formula used (see Chapter III., Vol. II.).

$\therefore$ Number of rivets to develop gross area of angle $= 2{\cdot}48f_c \div 0{\cdot}42f_t$ (*i.e.*, bearing)

$$= 2{\cdot}48 \times 0{\cdot}5f_t \div 0{\cdot}42f_t = 3 \text{ rivets}$$

Gross area of covers required $= 2{\cdot}48 + 5\%$, B.S.S. (4/22) $= 2{\cdot}60$ sq. in.

which results, of course, disregard the eccentricity of the load, unintentional or otherwise. The tension splice gives 5 rivets and 3·31 sq. in. gross of cover area and thus makes allowance for any eccentricity of loading, etc., because the fallacy in the foregoing calculation is that f_c is purposely reduced in value as the strut becomes longer so as to make allowance for the inherent bending stresses in a column.

Splices for Joists, Channels, etc.—Whether these sections act as tension members, compression members, or as beams, it will be found that if the net tensile strength of a section is developed by the splice covers and their rivets that the spliced member can act in any of these capacities. Further, it often happens that a design drawing indicates a splice, but neither the stresses nor the loads acting are given, so that the only satisfactory splice is that which develops the net tensile strength of the member.

As an example, the splice for an $8'' \times 6'' \times 35$ lb. R.S.J., Fig. 188, will be given here from the point of view of the development of the net tensile value, while in Chapter X. the same splice will be examined from the beam aspect by using equivalent moments of inertia, etc.

Gross area of joist		= sq. in.	10·3
Gross area of web	$= 8'' \times 0{\cdot}35''$	= ,,	2·8
Gross area of one flange	$= \frac{1}{2}(10{\cdot}3 - 2{\cdot}8)$	= ,,	3·75

Areas required.

Web covers, gross	$= 2{\cdot}8 + 5\%$, B.S.S. (4/22) $=$	sq. in.	2·94
Web covers, net	$= (2{\cdot}8 - 2$ holes @ $\frac{3}{4}'' \times 0{\cdot}35'') + 5\%$ $=$	,,	2·38
Flange cover, gross	$= 3{\cdot}75 + 10\%$, B.S.S. (4/22) $=$	,,	4·13
Flange cover, net	$= (3{\cdot}75 - 2$ holes @ $\frac{3}{4}'' \times 0{\cdot}648'') + 10\%$ $=$	,,	3·06

Areas given.

Web covers	2 @ $5\frac{1}{4}'' \times \frac{5}{16}''$, see Table 13 Vol. III; gross area $=$	,,	3·28
	net area $= 3{\cdot}28 - 4$ @ $\frac{3}{4}'' \times \frac{5}{16}''$ $=$	,,	2·34
Flange cover	1 @ $6'' \times \frac{11}{16}''$ per flange; gross area $=$	,,	4·13
	net area $= 4{\cdot}13 - 2$ @ $\frac{3}{4}'' \times \frac{11}{16}''$ $=$	,,	3·10

Rivets, $\frac{3}{4}''$ diameter.

Value per rivet in terms of f_t, which need not be known.

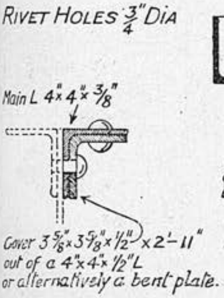
	Single Shear (S.S.) $= 0{\cdot}44 \times \frac{3}{4} f_t$ $=$	tons	$0{\cdot}33f_t$
	$\therefore$ D.S. $= 0{\cdot}66f_t$; and bearing on $0{\cdot}35''$ web $= 0{\cdot}35 \times \frac{3}{4} \times \frac{3}{2} f_t$ $=$	,,	$0{\cdot}4f_t$
Number required to develop net web covers	$= 2{\cdot}34f_t \div 0{\cdot}4f_t$, B. (and D.S.) $=$		6
Number required to develop net flange cover	$= 3{\cdot}10f_t \div 0{\cdot}33f_t$, S.S. (and B.) $=$		10

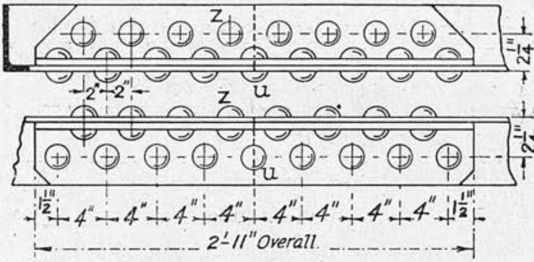
Checking the Joint.

Net strength of joist; tearing at CC	$= (10{\cdot}13 - 4$ @ $\frac{3}{4}'' \times 0{\cdot}648'') f_t$ $=$	tons	$8{\cdot}36f_t$
Net cover strength; tearing all four at AA	$= (2{\cdot}34 + 2$ @ $3{\cdot}10) f_t$ $=$	,,	$8{\cdot}54f_t$
Rivet strength; all rivets failing	$=$ web, 6 @ $0{\cdot}4f_t$ + flanges, 20 @ $0{\cdot}33f_t$ $=$	,,	$9{\cdot}00f_t$

Non-Symmetrical Angle Splice, Figs. 78 and 79.—The angle is $4'' \times 4'' \times \frac{3}{8}''$ with $\frac{3}{4}$-in. diameter rivets.

As the first of these figures shows (in heavy line and hatched cross-section) the rivets are in single shear and $\frac{3}{8}$-in. bearing. This joint is really an eccentric one with a B.M. acting in addition to the direct stress, the value of the moment being $P \times e$, where P is the stress in the bar and e is the eccentricity of the centre of gravity of the cover from the line of action of P. The load travels along the main angle from the right, and, near the joint, side-tracks through the rivets into the cover. It then travels the length of the cover which bridges the gap or joint, and finally returns to the left-hand portion of the main angle through the second group of rivets. To make allowance for this eccentricity of loading the B.S.S. (4/22) specifies that the cover shall be 10 per cent. in excess of the area covered. In the example the rivets which connect the cover to the





NON SYMMETRICAL JOINT OF A 4"x4"x3/8" ANGLE

FIG. 78. FIG. 79.

main angle will be given in sufficient number so as to develop this extra 10 per cent. strength of the covers.

The main angle may occur in the flange of a plate girder as indicated by the dotted portion of the cross-section. Due to the proximity of the web plate and flange plates, double covers, such as were used in the preceding case, cannot be employed. Further, no help is assumed to be given to the jointed angle by the adjacent elements of the structure other than the cover.

Occasionally round-backed angles are specified as covers. These angles are seldom rolled, and it is generally cheaper both in time and money to specify an ordinary angle with the heel machined to fit into the root of the main angle. Alternatively, a flat plate may be bent to suit.

The joint will be designed on the assumption that the main angle is in tension, as explained in the previous examples.

Gross cross-sectional area of $4'' \times 4'' \times \frac{3}{8}''$ angle	$= (4'' + 4'' - \frac{3}{8}'') \times \frac{3}{8}''$	= sq. in. 2·86

Net transverse area of angle $= 2{\cdot}86$ gross—1 hole $\frac{3}{4}'' \times \frac{3}{8}'' =$ sq. in. 2·58

Net diagonal area of angle as explained under $=$,, 2·18

Tensile strength of angle $=$ net area $\times$ working stress f_t $=$ tons $2{\cdot}18f_t$

Value of 1 rivet in S.S. ($\frac{3}{4}''$ dia.) $= f_s \times 0{\cdot}44 = \frac{3}{4} f_t \times 0{\cdot}44$ $=$,, $0{\cdot}33f_t$

Value of 1 rivet in $\frac{3}{8}''$ B $= f_b \times \frac{3}{8}'' \times \frac{3}{4}''$
$= 1\frac{1}{2} f_t \times \frac{9}{32}$ $=$,, $0{\cdot}42f_t$

Cover, net area required $= 2{\cdot}18$ sq. in. $+ 10\%$ $=$ sq. in. 2·40

Rivets required to develop the cover $= 2{\cdot}40f_t \div 0{\cdot}33f_t$ $=$ 8

Net Diagonal Tearing Area.—Refer to the key diagram of Table 10, p. 74 . Here P $= 2''$ and AB $=$ DE $= 1\frac{3}{4}''$, while $y = 2\frac{1}{4}'' + 2\frac{1}{4}'' - \frac{3}{8}'' = 4\frac{1}{8}''$. The gross length of B.D. is thus 4·584″. The net zigzag path for tearing of main angle is $(1\frac{3}{4}'' - \frac{3}{8}'') + (4{\cdot}584'' - \frac{3}{4}'') + (1\frac{3}{4}'' - \frac{3}{8}'') = 5{\cdot}817''$.

Whence the net zigzag area is $5{\cdot}817'' \times \frac{3}{8}''$ thick $=$ sq. in. 2·18.

Given :—1 Bent plate or machined angle $3\frac{5}{8}'' \times 3\frac{5}{8}'' \times \frac{1}{2}''$.

Gross cross-sectional area of cover $= (3\frac{5}{8}'' + 3\frac{5}{8}'' - \frac{1}{2}'') \times \frac{1}{2}''$ $=$ sq. in. 3·38

Net transverse area of cover $= 3{\cdot}38 - 1$ hole $\frac{3}{4}'' \times \frac{1}{2}''$ $=$,, 3·00

Net diagonal area of cover (as explained for main angle) $=$,, 2·60

Rivets on each side of joint, per leg $=$,, 4

Checking the Joint.

1. The rivets on one side of the joint failing by single shear, the lesser rivet value, *i.e.*, 8 rivets @ $0{\cdot}33f_t$ $=$ tons $2{\cdot}64f_t$
2. Main angle failing by zigzag tearing. Strength was $=$,, $2{\cdot}18f_t$
3. The bent cover plate tearing diagonally at ZU, Fig. 79 $=$,, $2{\cdot}60f_t$

Joint efficiency in tension $= 2{\cdot}18f_t \div 2{\cdot}86f_t = 0{\cdot}76$ or 76%

Riveting.—Note that the cover encroaches on the clearance for the rivet heads in the adjoining leg. Because of this the thicker the cover the smaller is the permissible rivet diameter. Alternatively the "standard" spacing of the rivet lines, *i.e.*, from heel to rivet line, could be departed from in the length of the splice.

Grouped and Reeled Splice, Figs. 80 to 83.—This form of splice is often used for splicing the main angles of plate girders. The practice in this country is to limit the length of the angles to about 40 ft. It may be valuable on certain occasions to have these angles over 40 ft. in length, but when over this length work is not facilitated in the shop. The practice in America is to specify that no splices will be permitted for the flange angles of plate girders if these can be obtained of the required length from the mills. Anent this, one American mill quotes "maximum length for all angles 100 feet."

No help is supposed to be given to the joint covers by the web or flange plates as these are working, presumably, at their full capacity. The joint has thus been shown without web or flange plates, so as not to confuse the reader. American practice seldom makes use of this type of joint, preferring the simpler one discussed in the previous article.

The angles are $6'' \times 6'' \times \frac{1}{2}''$, and each angle therefore has four rivet lines. The design of the girder fixed the rivet diameter at $\frac{15}{16}''$ finished size, in accordance with the recommendation of the B.S.S. (4/23, footnote). Design from the point of view of tension.

Rivet Values

Single shear $= 0{\cdot}7854\,(\frac{15}{16})^2 \times \frac{3}{4}f_t$ tons per rivet $= 0{\cdot}52f_t$
Bearing $\frac{1}{2}''$ Pl. $= \frac{15}{16}'' \times \frac{1}{2}'' \times \frac{3}{2}f_t$ tons per rivet $= 0{\cdot}7f_t$
Bearing $\frac{7}{16}''$ Pl. $= \frac{15}{16}'' \times \frac{7}{16}'' \times \frac{3}{2}f_t$ tons per rivet $= 0{\cdot}61f_t$

Rivet failure will thus take place due to single shear.

Net Area of One Main Angle

$$= 5{\cdot}75 \text{ sq. in.} - 2\,(\tfrac{1}{2}'' \times \tfrac{15}{16}'') = 4{\cdot}81 \text{ sq. in.}$$

Since at any cross-section of the angle only two holes occur.

Let V_1 = the total strength of the group of rivets in one plane between the joint and the end of one angle.
V_2 = ditto in leg of angle perpendicular to the foregoing leg.
V_3 = the same as for V_1, but in the other main angle.
V_4 to V_9 inclusive are then distributed in a similar manner.
c = the net area of each cover in sq. in.
a = the net area of each main angle = as above, 4·81 sq. in.

Case 1.—Failure by single shear should equal the strength of the main angles, Fig. 82.

$$\underset{\text{(Cover to main angle.)}}{(V_1 + V_2)} + \underset{\text{(Main angle to main angle.)}}{V_5} + \underset{\text{(Cover to main angle.)}}{(V_8 + V_9)} = \underset{\text{= original strength of main angles.}}{2af_t} \quad . \quad . \quad . \quad 1$$

If all the V's are equal, then $5V = 2af_t$

whence $V = \frac{2}{5}af_t \quad . \quad . \quad . \quad 2$

Case 2.—Both angle covers tearing and V_5 failing in single shear, Fig. 83.

$$cf_t + V_5 \text{ main angle to main angle} + cf_t = 2af_t \quad . \quad . \quad . \quad 3$$

$$\therefore cf_t = af_t - \tfrac{1}{2}V \quad . \quad 4$$

since $V_5 = V$, if the V's are the same.

Case 3.—Tearing straight through at a joint of the main angle as in Fig. 81. The metal torn will be on the line of rivets adjacent

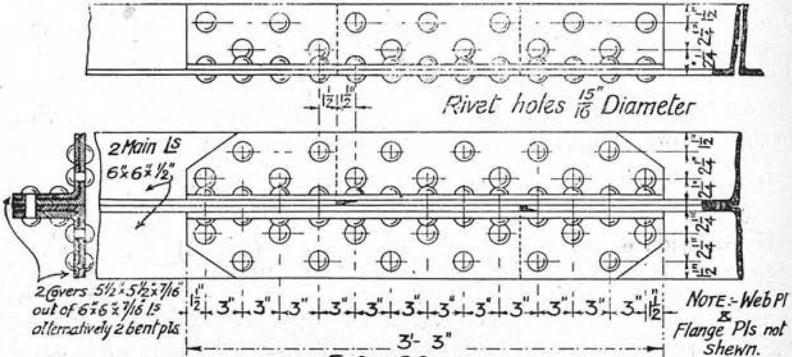


FIG. 80.

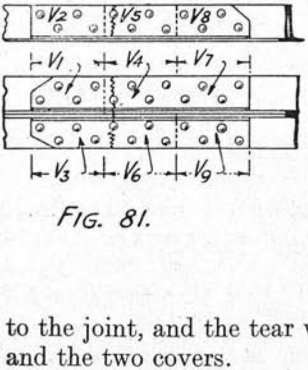


FIG. 81.

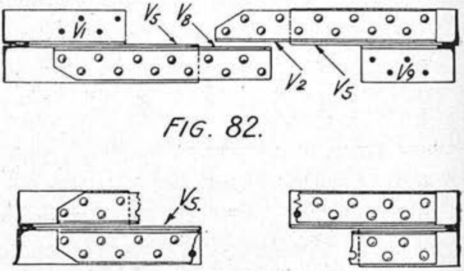


FIG. 82.

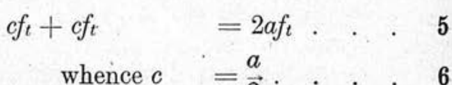


FIG. 83.

to the joint, and the tear will be through the remaining main angle and the two covers.

$$af_t + cf_t + cf_t = 2af_t \quad . \quad . \quad . \quad 5$$

$$\text{whence } c = \frac{a}{2} \quad . \quad . \quad . \quad . \quad 6$$

From **2.** $V = \frac{2}{5} \times 4{\cdot}81f_t$ and the number of rivets $= V \div 0{\cdot}52f_t = 4$ **7**

From **4** and **7**. $cf_t = 4{\cdot}81f_t - \dfrac{4 \times 0{\cdot}52f_t}{2}$

whence $c = 3{\cdot}77$ sq. in. . **8**

From **6.** $c = 2{\cdot}4$ „ . **9**

Two values are obtained for c because that of equation **4** depends partly upon the number of rivets used, while that of equation **6** depends entirely upon the area of the main angles.

The joint, considered as a whole, is symmetrical. Equation **9** gives the net cover area required, as it is independent of rivet values. Area required by the B.S.S. is 2·4 sq. in. + 5% = 2·52 sq. in.

Covers Adopted.—$5\frac{1}{2}'' \times 5\frac{1}{2}'' \times \frac{7}{16}''$. Net area of one = 4·62 − 2 holes $\frac{15}{16}'' \times \frac{7}{16}'' = 3{\cdot}8$ sq. in. The value for c in equation **8** is 3·77 sq. in. if 4 rivets per group are used. If the number of rivets per group be altered, 3·77 alters. The covers adopted thus satisfy both **8** and **9**, provided that at least 4 rivets per group are used.

Checking the Joint.—Substitute the values in the left-hand side of the following equations :—

In equation **1**.—5 groups of 4 rivets @ $0{\cdot}52f_t$ per rivet (Fig. 82) $= 10{\cdot}4f_t$
In equation **3**.—$3{\cdot}8f_t + 4$ @ $0{\cdot}52f_t + 3{\cdot}8f_t$ (Fig. 83) $= 9{\cdot}68f_t$
In equation **5**.—$4{\cdot}81f_t + 3{\cdot}8f_t + 3{\cdot}8f_t$ (Fig. 81) $= 12{\cdot}41f_t$
Net strength of two main angles $= 4{\cdot}81\,f_t \times 2 = 9{\cdot}6f_t$,

which has been developed. By increasing the mid-group of rivets from 4 to 5, *i.e.*, V_5 in equation **3** (and therefore also V_4 and V_6 since these reel with V_5), the strength rises to $9{\cdot}68f_t + 0{\cdot}52f_t = 10{\cdot}2f_t$. This has been done in the final detail of the joint, Fig. 80, in order to obtain a joint symmetrical in appearance. Compare Fig. 81 with Fig. 80.

The covers do not encroach too much upon the "fairway" on the inner side of the inner rivet line, and clearance is thus left for machine riveting. Had the cover been thicker, sufficient clearance between the inner rivet line and the side of the cover might not have been obtained. The standard rivet spacing, reading from the heel outwards, Fig. 80, is $2\frac{1}{4}$ in., $2\frac{1}{4}$ in. and $1\frac{1}{2}$ in., and this would hold good on both sides of the splice. At the splice the spacing could be altered to $2\frac{1}{2}$ in., 2 in. and $1\frac{1}{2}$ in., thus giving an extra $\frac{1}{4}$ in. between the inner rivet line and the heel, to accommodate the thick cover.

Minimum Reeled Pitch.—From what has been said in Chapter I. upon the cross-sectional area of angles, that of the $6'' \times 6'' \times \frac{1}{2}''$ angle of Fig. 68 is equivalent to a rectangle $11\frac{1}{2}''$ wide (not 12″) × $\frac{1}{2}''$ thick. In other words, the angle may be regarded as having been flattened out or developed. The two inner rivet lines will still be at the same distance from the toes of the angle, but will be 2 in. from the heel or centre line instead of $2\frac{1}{4}$ in.

If the angle fails by tearing it has four very probable paths :—(*a*) Straight across along the line ABDG ; (*b*) zigzag path ABCEF ;

(*c*) zigzag ABCDEF; and (*d*) ABCDG. The distances, centre to centre of holes, are given on the diagram.

The B.S.S. (4/4) specifies that only four-fifths of the *net* diagonal distances shall be taken in computing the effective area. The reason for this reduction factor was given in the previous chapter under the heading of Zigzag Tearing.

(*a*) ABDG.—Effective length $= 11{\cdot}5'' - 2 \ @ \ \frac{15}{16}'' \qquad = 9{\cdot}625''$

(*b*) ABCEF.—Effective length $= (1{\cdot}5'' - \frac{15}{32}'') + \frac{4}{5}(3{\cdot}75'' - \frac{15}{16}'') + 7{\cdot}75'' - 1\frac{1}{2} \times \frac{15}{16}'' \qquad = 9{\cdot}625''$

(*c*) ABCDEF.—Effective length $= 2\,(1{\cdot}5'' - \frac{15}{32}'') + 2 \times \frac{4}{5}(3{\cdot}75'' - \frac{15}{16}'') + \frac{4}{5}(5'' - \frac{15}{16}'') \qquad = 9{\cdot}8125''$

(*d*) ABCDG.—Effective length $= (1{\cdot}5'' - \frac{15}{32}'') + \frac{4}{5}(3{\cdot}75'' - \frac{15}{16}'') + \frac{4}{5}(5'' - \frac{15}{16}'') + (3{\cdot}75'' - \frac{15}{32}'') = 9{\cdot}8125''$

The tearing value of the main angles assumed in the calculations was the transverse area of $9{\cdot}625'' \times \frac{1}{2}'' = 4{\cdot}81$ sq. in. and agrees with (*a*) and (*b*). The reeled pitch of 3 in. is the minimum reeled pitch for transverse tearing to occur. Had the holes been 1 in. diameter the respective lengths would be 9·5 in., 9·45 in., 9·6 in. and 9·6 in., *i.e.*, diagonal tearing along ABCEF giving a net area of $9{\cdot}45'' \times \frac{1}{2}'' = 4{\cdot}725''$. Similarly, had the reeled pitch been $2\frac{1}{2}$ in. instead of 3 in. for the $\frac{15}{16}$-in. diameter holes, the effective diagonal area would have been less than the transverse area.

Compare these results with the net areas as given in Table 11, Vol. III., which has its key diagram lettered to correspond with the foregoing text.

Diagonal Tearing of the Cover.—The width of the $5\frac{1}{2}'' \times 5\frac{1}{2}'' \times \frac{7}{16}''$ cover angle when developed out is $2 \times 5\frac{1}{2}'' - \frac{7}{16}'' = 10\frac{9}{16}''$.

(*a*) ABDG.—Effective length $= 10\frac{9}{16}'' - 2 \ @ \ \frac{15}{16}'' \qquad = 8{\cdot}69''$

(*b*) ABCEF.—Effective length $= (1{\cdot}5'' - \frac{15}{32}'') + \frac{4}{5}(3{\cdot}75'' - \frac{15}{16}'') + 6\frac{13}{16}'' - 1\frac{1}{2} \times \frac{15}{16}'' \qquad = 8{\cdot}69''$

(*c*) ABCDEF.—Effective length $= 2\,(1{\cdot}5'' - \frac{15}{32}'') + 2 \times \frac{4}{5}(3{\cdot}75'' - \frac{15}{16}'') + \frac{4}{5}(4{\cdot}25'' - \frac{15}{16}'') \qquad = 9{\cdot}21''$

(*d*) ABCDG.—Effective length $= (1{\cdot}5'' - \frac{15}{32}'') + \frac{4}{5}(3{\cdot}75'' - \frac{15}{16}'') + \frac{4}{5}(4{\cdot}25'' - \frac{15}{16}'') + (3{\cdot}75'' - \frac{15}{32}'') = 9{\cdot}21''$

The tearing value of the cover angle was assumed to be the transverse area of $8{\cdot}69'' \times \frac{7}{16}'' = 3{\cdot}8$ sq. in., which is in keeping with the above calculations.

Other Types of Flange Angle Splices.—As pointed out when dealing with the angle splice of Fig. 80, it often happens that the cover angle required is so thick that the rivet heads cannot be closed with standard machine dies. Dies are sometimes ground down so that clearance is obtained, but this may result in a weak rivet head. Another alternative is to depart from the standard rivet lines for the length of the joint and make the distance, heel of angle to centre of rivet, greater than the standard suggests.

It often arises, especially with small-legged thick angles and angles with double rivet lines, that a change of position of the rivet line brings the rivets too close to the edge of the section. When this occurs Fig. 84 shows a method of lowering the bent cover-plate thickness by placing part of the cover area in the form of a flat bar next to the unbroken main angle.

Only one angle is being jointed at this particular part, viz., that under the bent plate. The total area of the covers is the sum of the vertical flat and the bent plate cross-sectional areas. Where the thicknesses are the same the ratio of the gross areas of metal of the two covers is approximately 1 : 2 respectively. The thickness of the

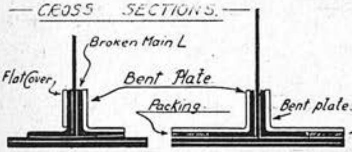


FIG 84. FIG 85.

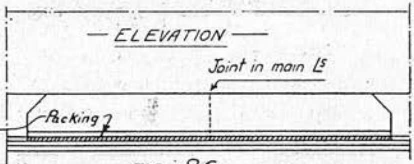


FIG. 86.

vertical plate rarely exceeds that of the bent plate and is usually less.

The rivets through the horizontal leg are, in accordance with the premise, in single shear and bearing, while those through the vertical leg—there being two vertical covers—are in double shear and bearing.

The broken main angle is unsymmetrically covered. It is also unsymmetrically developed by the rivets, since the vertical rivets are in double shear and those through the horizontal leg are in single shear. Extending the covers so as to include an extra rivet or two would therefore not be amiss.

Figs. 85 and 86 show a type of joint which is used where a straight through break of the two main angles occur. It can also be used when one angle only is broken, or it can be used in the grouped and reeled angle splice of Fig. 80. The bent plates are carried right out to the edge of the flange plates and are riveted to them through the packings. These latter rivets should not be calculated as carrying any load from the main angles, but be merely looked upon as stitching or tacking rivets. The effective rivets are those through the main angles.

This joint, with kneed and plated stiffeners placed at or near the break of the main angles, gives lateral stiffness. It is, therefore, a good joint to use in the case of crane girders, whose flanges have often to act as horizontal girders in resisting the dragging of weights across the shop floor in a direction at right angles to the crane girder. Crane girder compression flanges are generally unsupported extraneously in a lateral direction, and any extra strength and rigidity at a flange angle joint is therefore worthy of consideration.

Just as the rivets through the packings should not be considered to take any load, so also with a portion of the metal of the bent covers. Perhaps a fair assumption would be to say that the effective cover metal is that in direct contact with the flange main angles plus one-half of that in contact with the packings.

The reason is similar to that given in Chapter VIII. relative to the net effective area of a tension angle with an unriveted leg, as illustrated in Fig. 136.

Figs. 87 and 88, representing one main angle broken and two

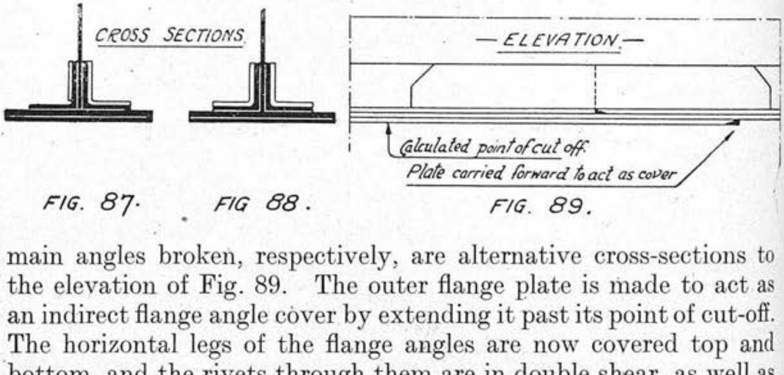


FIG. 87. FIG 88. FIG. 89.

main angles broken, respectively, are alternative cross-sections to the elevation of Fig. 89. The outer flange plate is made to act as an indirect flange angle cover by extending it past its point of cut-off. The horizontal legs of the flange angles are now covered top and bottom, and the rivets through them are in double shear, as well as the rivets through the vertical legs.

The bent plate covers of Fig. 85 can also be used with the extended or prolonged flange plate.

If the main angles can be broken near the point of cut-off of the flange plate this joint should be used, as it lessens the thickness of the bent plate covers. On the other hand, there is often nothing (except appearance) to hinder the designer from placing the main angle joints where he pleases, and using an additional outer cover plate on the flange. This small " flange " plate would only run the length of the joint, but in exposed work of some types would appear rather unsightly.

Rivets in Joints doing Double Duty.—Sometimes it is asked " Why are rivets which are used to transfer the shear from the web plate into the main flange angles, and which are supposedly already working at their full capacity, also called upon to carry an extra load due to an angle splice ?" The answer is that the rivets are not in single or double shear, but may be in treble or even quadruple shear.

Fig. 90 shows a rivet which, carrying a load of 12 tons, is apparently dangerously overloaded. On investigating the shearing

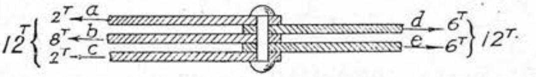


FIG 90.

forces it is found, however, that the single shear at any one section is well within the permissible value.

1″ dia. rivet. Single-shear value $= 0{\cdot}7854 \times 6^{\text{T}}$
with $f_t = 8^{\text{T}}$/sq. in. $= 4{\cdot}71$ tons

At the common face of plates a and d the shear is $= -\,2$,,

At the common face of plates d and b the shear is
$-\,2 + 6$ $= +\,4$,,

At the common face of plates b and e the shear is
$-\,2 + 6 - 8$ $= -\,4$,,

At the common face of plates e and c the shear is
$-\,2 + 6 - 8 + 6$ $= +\,2$,,

The maximum single-shear value of 4 tons is under the permissible value of 4·7 tons. When cover plates are riveted to the outside of main angles the extra load brought on to the rivet is applied, as a single shear, on the common surface between the added cover and the outer face of the main angle, and the permissible shearing value is not exceeded.

Bearing Value.—The pitch of the rivets through the flange main angles would be, in all probability, settled by the bearing value of the rivets on the web plate.

As a rule the web plate is thinner than either of the main angles, and certainly thinner than their added thicknesses.

If, therefore, the bearing value of the rivet is safe with the web, it is still more safe on the main angles, and safer still at the main angle covers of a joint, since these are always at least 5 per cent. thicker than the main angles.

CHAPTER VII

SPLICES FOR THE WEB PLATES OF PLATE GIRDERS

Length and Thickness of Web Plates.—The depth of the web plate of a plate girder is generally about four times the width of the flange plates, and, therefore, web plates cannot be obtained in such long lengths from the mills. Even if they could be obtained they would prove to be too heavy and awkward to handle in the shops. The web plate, if over 30 ft. or thereby in length, is usually built up of two or more lengths butted together and the joint covered with a splice plate on each face of the web. Further, the presence of a splice permits of a camber being given to the girder which would otherwise be difficult to obtain if the plate were in one piece (see the article on Camber, Chapter I, Vol. II).

In heavy girders of large span it is economical to vary the web thickness twice or thrice, and the web splices permit of this alteration of section.

The following table may be of service to the beginner, but it must be understood that it is only a rough guide. The number of joints, or splices, will be one less in number than the number of pieces forming the length of the web plate.

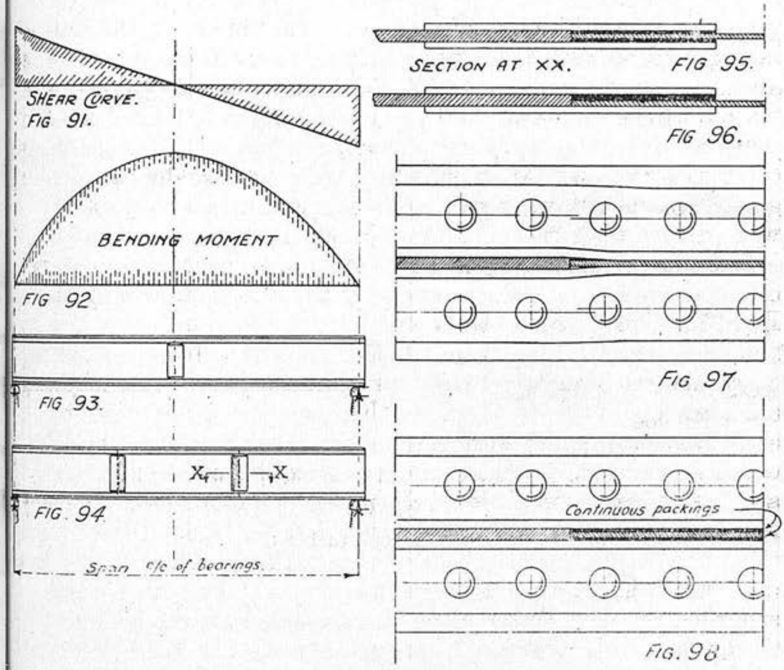
Overall Length of Girder in Feet.	Thickness of Web Plate.	Number of Pieces forming Web.
About 30—35	Same throughout	One (or two).
" 40	" "	Two.
" 50	" "	Two (or three).
" 60	" " if light	Two, three or four.
" 60	Two different thicknesses if heavy .	Three or four.
" 70	Same throughout if light	Three or four.
" 70	Two (or three) different thicknesses if heavy	Three, four (or five).

The term "light" is used for web plates up to and including $\frac{1}{2}$ in. thick.

The term "heavy" is used for web plates including and over $\frac{3}{4}$ in. thick.

Web thicknesses between $\frac{1}{2}$ in. and $\frac{3}{4}$ in. thick would fall either into the light or heavy category, according to the design and the designer.

The position of the web splice depends upon the limiting size of the plate rolled (see Table I., Vol. III.), and perhaps also upon the position of extraneous connections to the girder. In a crane girder there is practically no restriction upon the position of the web splice, and the same may be said for a deck span bridge. In a through span



bridge with cross girders the splice must not occur at the junction of the cross to the main girder. Its position is therefore fixed by symmetry midway between two cross girders. Further, splices should always be placed so that a symmetrical appearance is obtained. The reason is partly æsthetic and partly economic. The latter reason will be apparent from Fig. 94, since the outer left-hand portion is similar but "other hand," *i.e.*, a reflection, to the outer right-hand portion. In this way templates, etc., are reduced in number and, therefore, also the cost of fabrication.

Diagrams 93 and 94 show alternative schemes for a girder which carries a uniformly distributed load. In the former diagram, since there is only one web splice, the web plate, although in two portions, must be of the same thickness throughout. In the latter diagram the web plate is shown as being in three pieces, so that the mid portion may be thinner than the two end pieces. This follows, of course, from the fact that the shear is much less in the mid portion than it is in the abutment portions.

Web Packings, Figs. 95 and 96, are shown inked in and are placed under the web plate covers on the thin mid portion of the web. These covers extend from main angles to main angles. If it is desired to have the longitudinal centre line of the girder coincident with the plane of loading and of bending, then thin packings will be required on each face of the thin portion of the web plate as illustrated in the first figure. The alternative to this method is to place a packing of twice the thickness on only one side of the thin portion of the web plate after the manner of Fig. 96. The resulting eccentricity is slight and is usually neglected. The advantage gained is that a thicker packing is obtained which is more easily worked with than the two thinner packings.

Springing of Main Angles and Packings.—The main angles are often sprung from the thick portion of the web to the thin portion, Fig. 97. This is accomplished by using the high-pressure riveting machines for clenching them together, and the rivets contracting on cooling permanently exert this required force. The resulting axial tension on the rivets exists only on these rivets adjacent to the set. No special smith work is necessary unless the main angles are exceptionally heavy. This springing of angles is quite common, and another example is met with in the rafters of a roof truss. In many roof trusses the shoe and apex gusset plates are made thicker than the remaining gussets attached to the rafter bars. The rafter angles on leaving the shoe plate, therefore, come close together, so as to sandwich the next gusset plate. In this example the springing of the angles is more gentle than is the case with the plate girder main angles. The disadvantage of springing the main angles is that it upsets the rivet lines in the flange plates, as the rivets will not lie on continuous longitudinal straight lines. Conversely, if they be kept collinear in the flange plates, then at that portion where the web is thin the rivets must approach the toes of the main angles, Fig. 97.

Fig. 98 indicates how the foregoing disadvantage is removed. A continuous thin packing (the same width as the main angle leg with which it is in contact) is placed under each main angle, where these are riveted to the thin portion of the web. The main angles are now absolutely straight throughout their full length. This method is used in conjunction with that of Fig. 95.

Alternatively, the packing may be doubled in thickness and placed on one side of the web only. Both main angles will thus again be straight throughout their full length. This latter method will be used in that girder where the web splice of Fig. 96 is adopted. Here again the plane of loading is no longer coincident with the longitudinal vertical plane of the girder. The slight eccentricity of the web plate will cause a secondary bending moment, but not of large amount.

From the foregoing remarks it will be seen that the thinning down of the web plate does not always result in a financial gain.

Comparison of Web and Flange Splices.—Owing to the fact that a web splice is generally placed where the shear is not large, two methods of designing a web splice are :—

(1) Designing for the actual stress occurring at the joint.

(2) Designing the splice so that it is as strong as the web plate covered. The splice thus obtained is usually overstrong.

Compare this with the flange splices. When the bending moment decreases less metal is required in the flanges, and one flange plate is cut off (Chapter X .). This cutting off of the flange plates is repeated until, in the case of the tension flange of a simply-supported girder, there is often only the main angles left near the abutments where the bending moment approaches zero. The same accuracy of grading the metal, according to the stress, is not obtainable in the web. The angles and plates of the flanges are thus practically always worked up to their full capacity, and when these are spliced the covers must develop fully the strength of the parts covered.

The B.S.S. (4/22) limits the design of the web splice to either a combination of (1) and (2), or wholly to the second method. The paragraph states that the sectional area of the covers shall be at least 5 per cent. in excess of the area covered. If the web plate is being carried throughout at the same thickness the cover plates will be overstrong, since the web plate itself is overstrong at the joint, which is usually situated at a point of minimum shear. The rivets, however, have only to be of a sufficient number to develop the stress and not the full strength of the part covered.

The choice is thus left to the designer of making the rivets develop the stress, or, by using more rivets, develop the section covered. The same point was discussed with regard to the two end groups of rivets in the outer flange-plate cover of Fig. 75.

Two Methods of Calculating Web Splices.—(1) When the flanges of the girder are assumed to take all the bending moment and the web is supposed to take only the shear, then the splice is designed for shear only.

(2) When the web aids the flanges in carrying bending moment, then the web splice must be designed to carry both the total shear

at the section and that proportion of the total bending moment allotted to the web.

Simple Web Splice for Shear only.—In this case the web covers and their rivets will, therefore, offer the necessary resistance to the shear; any help which the flanges may afford is disregarded. In Fig. 99 the right-hand portion of the web can only sink relatively to the left :—(*a*) If all the rivets through the web cover plates on one side of the joint fail by crushing (bearing) or in double shear, Fig. 101.

(*b*) If the web covers have their vertical cross-sectional gross area sheared through.

In Fig. 100 the metal to be cut through is $39\frac{3}{4}'' \times \frac{5}{16}'' \times 2$. The B.S.S. (4/4) states that, "The shearing stress on the web plates of plate girders shall be calculated on the gross sectional area of the full depth of the plate." This is practically equivalent to saying that the rivet shanks fill the holes in the web covers and the metal to be sheared through is the gross area and not the net area. The reasoning is the same as was used for compression members. Some designers deduct the area of the rivet holes in arriving at the effective resisting area of the web covers, but the working stress is correspondingly higher than that of the B.S.S., which adopts, as the permissible shearing stress on gross area, five-eighths of the working tensile stress, B.S.S. (3/18).

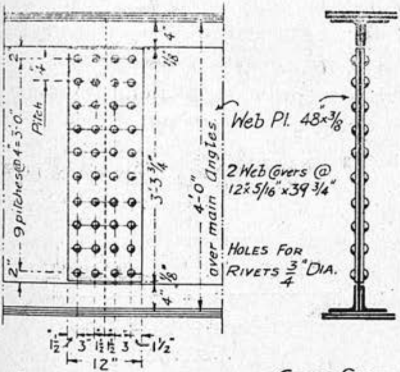
It must be particularly noted that the plate web girder is not the simple structure our calculations would make it appear to be. It is extremely complex, and because of this the calculations used by the designers are really approximations, and being approximations must vary, since no clean-cut arithmetical solution can be obtained. By their specification the British Standards Committee virtually have laid down the method of calculation.

Comments on Simple Web Splices.—*Always give at least two rows of rivets on each side of the joint* although calculations only ask for one. The vertical rivet pitch can be widened out, but must not exceed the maximum permissible pitch of twelve times the thinnest plate. Two rows of rivets give a fixity or rigidity to the cover plates, and therefore to the whole joint, in a transverse direction, that is lacking where there is only one row of rivets.

In agreement with the assumption that the web takes shear only, the minimum pitch is limited to $3d$, and not to any consideration of lowering the web plate's section modulus, as happens in the case of the web which takes bending moment in addition to shear.

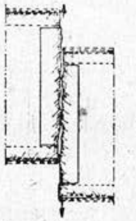
Text-books on "Strength of Materials" show that the distribution of the shear stress on the web, *between* the top and bottom main angles, is fairly uniform. The vertical pitch can thus be made uniform, and every rivet will carry approximately the same load. This is not the case with a web splice which also carries bending moment.

Students when given a uniform load on a girder often conclude that the maximum shear at a splice is that obtained from the shear diagram of Fig. 91. This is true if the load is constant and if when first applied to the girder it is distributed evenly throughout. If one end is loaded before the other this shear curve no longer holds. In this case it is equivalent to a uniform live load advancing on to the span from either end. There must therefore be a shear, some time or other, at the girder centre line. To allow for all contingencies



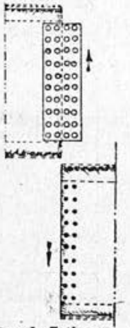
— Elevation — — Cross Section —

— Simple Web Splice — Fig. 99.



— Covers Tearing —

Fig. 100.



— Rivet Failure —

Fig. 101

which may arise, it is better—whether the load is dead or live—to find the maximum possible shear at any splice by loading the girder between the splice and the furthest away abutment, and to design the web splice for this shearing force.

Example of Web Splice for Shear, Fig. 99.—*Given* :—Girder 4 ft. deep over angles, with $\frac{3}{8}$ in. thick web plate throughout : shear at splice 50 tons ; finished rivet diameter $\frac{3}{4}$ in. ; the permissible stresses to be those of the B.S.S. (3/18) and (4/23).

Rivet Values.

Double shear $= 0{\cdot}44$ sq. in. $\times$ 12 tons/sq. in. $= 5{\cdot}30$ tons/rivet

$\frac{3}{8}''$ bearing $= \frac{3}{8}'' \times \frac{3}{4}'' \times 12^{T}/$ sq. in. $= 3{\cdot}38$,,

Number of rivets required $= 50^{T} \div 3{\cdot}38^{T} = 15$,

on each side of joint.

Number of rivets adopted to obtain an even pitch $= 20$

Cover Area.

Gross area of web pl. $= 48'' \times \frac{3}{8}''$ $= 18$ sq. in.

Gross area of covers $= 18$ sq. in. $+ 5\%$ required [B.S.S. (4/22)] $= 18{\cdot}9$,,

Gross area of covers $=$ that of 2 @ $39\frac{3}{4}''$ given $\times \frac{5}{16}''$ $= 24{\cdot}8$,,

[$\frac{1}{4}''$ too thin for outside work, see "Minimum Sections," B.S.S. (4/6).]

In order that the cover plates may clear any sectional growth of the main angles, $\frac{1}{8}$ in. is allowed, top and bottom, as a clearance.

Alternative Method.—If the shear were unknown the safest

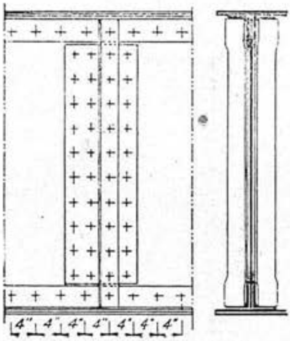


FIG. 102.

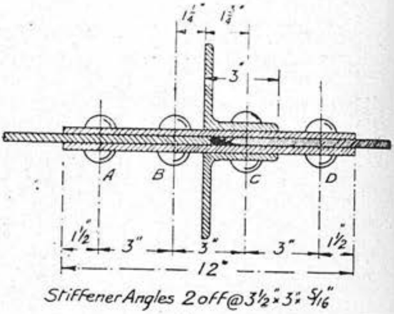


FIG. 103.

method would be to design the splice so as to develop the full strength of the web plate.

From the B.S.S. (3/18) the permissible shearing stress on the web $= f_w = \frac{5}{8} f_t$

Bearing value of 1 rivet $= \frac{3}{8}'' \times \frac{3}{4}'' \times \frac{3}{2} f_t$ $= 0{\cdot}42 f_t$ tons

Shear value of the web pl. $= 48'' \times \frac{3}{8}'' \times \frac{5}{8} f_t$ $= 11{\cdot}25 f_t$,,

Number of rivets required $= 11{\cdot}25 f_t \div 0{\cdot}42 f_t$ $= 26{\cdot}8$

This would give two rows of 13 rivets per row at 3 in. vertical pitch, and 2 in. from the centres of the outermost rivets to the toes of the flange angles.

Figs. 102 and 103.—It is good practice to have a double angle stiffener placed over a web splice wherever possible. It gives rigidity to the structure where it is most needed, and is given as an additional safeguard.

Fig. 103 is an enlarged cross-section indicating the position of the heels of the stiffener angles relative to the joint. The rivets through the web covers were pitched at 3 in. horizontally. Rivet C has in addition to pass through the stiffener angle, whose rivet line for the 3-in. leg is $1\frac{3}{4}$ in. from the heel. The heels of the stiffener angles therefore take the positions marked on the drawing. There is no necessity for the angle heels and the joint to be collinear. The distance of $1\frac{1}{4}$ in. from rivet B to the heel of the stiffener gives sufficient clearance for the rivet die to close rivet B.

Fig. 104 is a magnified view of the upper portion of Fig. 102. The stiffener angles are joggled over the vertical legs of the main angles and are ground down so as to fit tightly against the horizontal legs of the main angles, B.S.S. (2/3). Rivet X must be kept far enough away from the higher metal of the riveted leg in order to allow the standard die to close rivet X. This distance of 2 in.

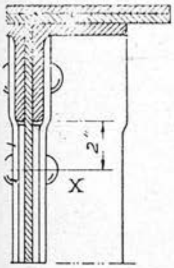


FIG. 104.

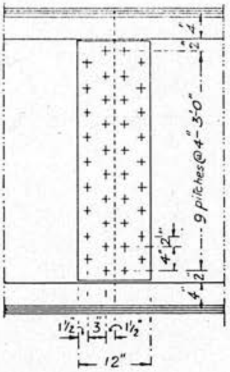


FIG. 105.

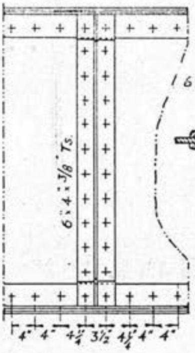


FIG. 106.

FIG. 107.

was the governing dimension in laying out the vertical pitching of the cover rivets of Figs. 99 and 102. The former figure, for clearness, is shown without stiffeners.

Fig. 105 is an alternative detail to that of Fig. 99. The rivets are reeled vertically in order that at any horizontal section through the rivets there are only two holes out of the cover plate instead of the four of Fig. 99. The resulting joint is assumed by some designers to be stronger on this account, but this is questionable owing to the inclined tensile and compressive forces existing in the web plate. Reeled holes are on this account more apt to facilitate than to prevent collapse through diagonal fracture from hole to hole. It is a trifle cheaper to have the chain riveting of Fig. 99 than the reeled riveting of Fig. 105, but there is little to chose between them either on the score of strength or expense.

Figs. 106 and 107 illustrate a web splice which is still used, but fortunately by few designers. Little can be said in its favour.

The primary idea is economy in so far that the tee acts sim taneously as a web cover for the joint and as a stiffener.

The use of this section is at variance with the recommendati of a previous paragraph, wherein it was stated that a web spl should have at least two rows of rivets on each side of the joint f lateral stiffness.

Further, the tee itself is a bad section to use as a stiffener on plate girder. The standard spacing of the parallel lines of rive in the table of the tee is generally different from the pitch the main angle rivets. The main angle rivet pitch is theref broken in its continuity by the tee. In that particular portion the girder which is drawn the standard pitch is 4 in., but t introduction of the tee section necessitates a larger pitch of $4\frac{1}{4}$ i then $3\frac{1}{2}$ in., and again $4\frac{1}{4}$ in. in order that their sum of 12 in. be t same as three standard pitches of 4 in.

Compare this with Fig. 102, where the angle stiffener does n interfere with the continuity of the pitch of the main flange an rivets.

Example of a Web Splice for B.M. and Shear. Figs. 108 and 1

Data.—Design the web splice for the 40-ft. span gantry girder Chapter II. Vol II. Web plate $48'' \times \frac{3}{8}''$, with one-eighth of its a included in each of the flange areas. Tension flange net area 17·2 sq. in., made up thus :—$\frac{1}{8}$ web = 2·25, angles = 9·62 a flange plate = 5·3 (item **16** of the girder calculations). To maximum bending moment at the section is 479·7 ft. tons, with simultaneously occurring shear of 40 tons (see items **42** and **43** the girder calculations). Finished rivet diameter $\frac{15}{16}$ in. The pe missible stresses to be those of the B.S.S. (3/18, 4/9 and 4/23).

The term " one-eighth of the web " represents the approximati made as to the net area of the web which is effective as tensi flange area (see Chapter .). For this reason, and the followi one also, the joint will be designed from the aspect of tension, a the compression side of the cover will be made similar to the tens side.

In the splice the upper edges of the web plate tend to meet, a this butting action relieves the rivets and covers of stress, the cove simply functioning as guides. The lower edges of the joint tend leave each other, thus placing stress on the cover rivets, while t bottom covers themselves will be in tension.

$$\frac{\text{B.M. carried by web}}{\text{B.M. total}} = \frac{\frac{1}{8}\text{ web area}}{\text{total tensile area}} = \frac{2\cdot25}{17\cdot2} = 0\cdot131.$$

∴ B.M. carried by the web = 0·131 of total B.M. of 479·7 $\begin{cases} = \text{ft. tons } 62{\cdot}7. \\ = \text{in. tons } 753. \end{cases}$

Since the edge angles of the top flange are assumed to form an independent horizontal girder, the centre of gravity, and therefore the neutral axis of the vertical plate girder, is midway between the flanges, as is the case in most plate girders.

The permissible tensile stress at B, the centre of gravity of the flange, is 8 tons per square inch. A fibre at D will have a much lower stress acting on it since it is nearer the neutral axis, *i.e.*, nearer the axis of no stress. Stress at D = that at B $\times \dfrac{\mathrm{D}d}{\mathrm{B}b} =$

$8 \times \dfrac{\mathrm{OD}}{\mathrm{OB}} = 8 \times \dfrac{13\frac{1}{2}}{23} = 4{\cdot}7$ tons per square inch. This is the average stress acting in cover D.

Similarly, if the D.S. and rivet-bearing values on $\frac{3}{8}$ in. web at B

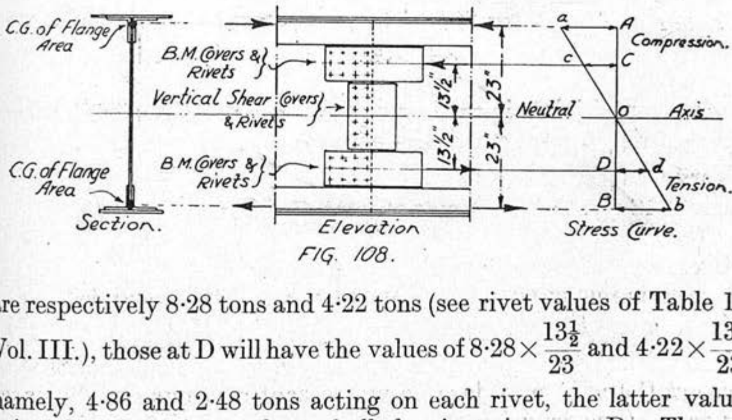


FIG. 108.

are respectively 8·28 tons and 4·22 tons (see rivet values of Table 12, Vol. III.), those at D will have the values of $8{\cdot}28 \times \dfrac{13\frac{1}{2}}{23}$ and $4{\cdot}22 \times \dfrac{13\frac{1}{2}}{23}$, namely, 4·86 and 2·48 tons acting on each rivet, the latter values being the mean rivet values of all the rivets in cover D. The row nearer the neutral axis will have a lesser value, and the outer row a greater value than the mean.

The magnitude of the tensile force in covers D is equal to that of the compressive force in covers C. These two forces form a couple, whose lever arm is twice $13\frac{1}{2}$ in., counteracting the B.M. in the web.

∴ Force C = Force D	= 753 in. tons ÷ 27″	= tons	28
Number of rivets required in each set of covers	= 28 ÷ 2·48	= 11·3, *i.e.*,	12
Area required for tensile covers	= 28 ÷ 4·7	= net sq. in.	6
Given 2 pls. @ 9″ × ½″, area net	$= 2\,(9'' - 3 \times \frac{15}{16}'') \times \frac{1}{2}'' =$	,,	6·2

The compression or upper covers are made similar to the tensile ones.

Mid Vertical Covers.—The rivets in these are calculated on the assumption that they carry only vertical shear, and since shear is practically constant or uniform throughout the girder depth, the full rivet value of 4·22 tons is used. As will be seen from the following alternative design, these centre rivets must also partake of the bending moment, and therefore this method of calculation is not theoretically correct. Further, the combined web splice has

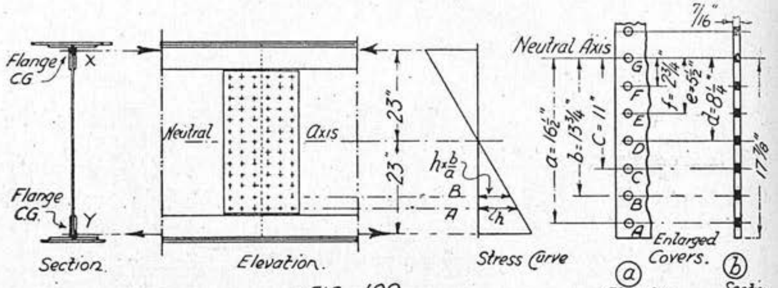


FIG 109.

two horizontal joints in the cover plates, where the horizontal shear is not carried across into the adjoining cover.

Vertical shear at section		= tons	40
Given 12 rivets on each side of joint @ 4·22 tons (bearing $\frac{3}{8}''$)		= ,,	50·6
Gross area of covers required = vertical shear ÷ permissible stress	$= 40 \div 5$	= sq. in.	8
Gross area of covers given	$= 2 \ @ \ 17\frac{3}{4}'' \times \frac{1}{2}''$	= ,,	$17\frac{3}{4}$

The thicknesses of the inner vertical covers are kept the same as those at C and D in order to prevent double joggling of the angle stiffeners at the joint.

Alternative Method for a B.M. and Shear Web Splice, Figs. 109 and 111.

Data.—As for previous example.

In this case the cover on either side of the web is in one piece and not in three. The rivets are, therefore, subjected to a horizontal pull due to bending plus a vertical load due to shear. Where the web carries B.M. as well as shear, it is always the former which controls the design. In fact, no great harm would occur in the majority of cases were shear to be neglected altogether.

At Y, the centre of gravity of the bottom flange, the permissible tensile stress is 8 tons per net square inch. The working loads per rivet at Y are :—D.S. = 8·28 tons and $\frac{3}{8}$ in. bearing on web = 4·22 tons. From the stress curve the tensile stress at $17\frac{7}{8}$ in. from the neutral axis (*i.e.*, the extreme fibre of the cover) is $8^{T} \times 17\frac{7}{8}'' \div 23'' =$ 6·2 tons/sq. in. Similarly, the working load on rivet A must not exceed $4{\cdot}22$ tons $\times\ 16\frac{1}{2}'' \div 23'' = 3{\cdot}03$ tons.

The web tends to open out at the bottom due to the bending action, but is restrained by the covers. The covers pick up their load from the rivets, and therefore rivet A offers a resisting moment about the neutral axis, the value of which is the actual

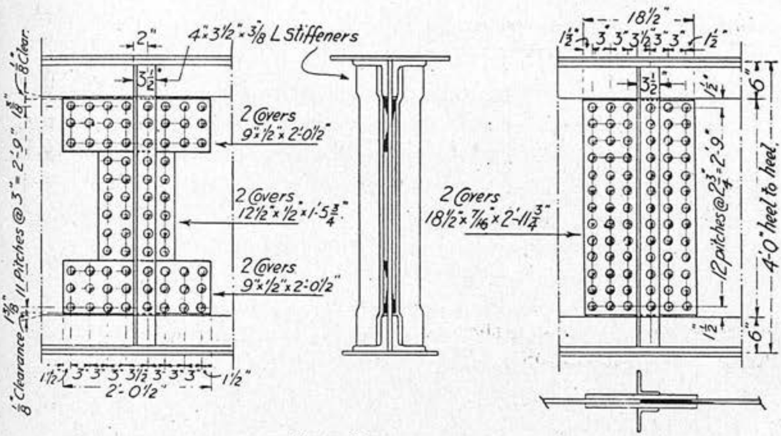


FIG. 110. HALF SECTIONS OF FIGS 110 & 111. FIG. 111.

load on rivet A multiplied by lever arm a. Similarly, rivet B offers a resisting moment of rivet load at B multiplied by its lever arm b, and so on.

Let the horizontal load on rivet A, due to bending, be h tons, then that on rivet B is $h \times (b \div a)$ tons, and that on rivet C is $h \times (c \div a)$, etc. ; *i.e.*, proportional to the rivet distance from the N.A. as shown on the stress curve above ; or, in tabular form :—

Rivet.	Actual Horizontal Load.	Lever Arm.	Moment.
A	h	a	$ha = ha^2 \div a$
B	$h \times (b \div a)$	b	$hb^2 \div a$
C	$h \times (c \div a)$	c	$hc^2 \div a$
D	$h \times (d \div a)$	d	$hd^2 \div a$
		etc.	

The total resisting moment of one half row

$$= \frac{h}{a} \Sigma a^2 + b^2 + c^2 + d^2 + \ldots + g^2.$$

For one complete row the moment is $\frac{h}{a} \times 2\Sigma a^2 + b^2 + \ldots + g^2$. The numerical value of this (remembering that h should not exceed 3·03 tons) is $\frac{3\cdot03}{16\frac{1}{2}} \times 2\,[16\frac{1}{2}^2 + 13\frac{3}{4}^2 + \ldots + 2\frac{3}{4}^2 + 0^2] = \frac{3\cdot03}{16\frac{1}{2}}\,[1376\cdot36] = 253$ in. tons : see Figs. (*a*) and (*b*).

In order to increase the resisting moment of the rivets these were pitched vertically at the minimum spacing of $3d = 2\frac{3}{4}''$. See further remarks on this point in items **44** and **46** of the main girder calculations.

Bending moment carried by web	= in. tons	753
∴ Number of rows of rivets required on each side of joint = 753 ÷ 253	=	3
Vertical load per rivet due to shear = v = 40 tons ÷ 39 rivets	= tons	1

Since the B.M. of 753 = 3 rows at $\frac{h}{a} \times$

$$2\Sigma a^2 + b^2 + \ldots + g^2 = \frac{h \times 3}{16\frac{1}{2}}\,(1{,}376\cdot36)$$

∴ The horizontal load on rivet A = h = $753 \times 16\frac{1}{2} \div 3\,(1376\cdot36)$	= tons	3
By the parallelogram of forces, resultant load on rivet A = $\sqrt{h^2 + v^2} = \sqrt{10}$	= tons	3·16

Permissible load on rivet A = 3·03 tons. The overload on rivet A is only 4 per cent. and may be neglected in view of the excess flange material which exists at this point. See item **47** of main girder calculations.

Thickness of Cover Plates.—The first trial proved $\frac{3}{8}$ in. thick plates to be too thin. Try, therefore, two plates at $35\frac{3}{4}'' \times \frac{7}{16}''$, thus giving $\frac{1}{8}$ in. clearance top and bottom.

Gross moment of inertia of one plate, Fig. *b* $= \frac{1}{12} \times \frac{7}{16}'' \times (35\frac{3}{4}'')^3$	= in.⁴	1666
Inertia of holes = Σ (area) × (distance)² $= (\frac{15}{16}'' \times \frac{7}{16}'')\,[2(16\frac{1}{2}^2 + 13\frac{3}{4}^2 + \ldots + 2\frac{3}{4}^2 + 0^2)]$ $= \frac{15}{16}'' \times \frac{7}{16}'' \times 1376\cdot36$	= in.⁴	564
Net moment of inertia I of one cover	= ,,	1102

Net moment of inertia I of two covers, back and front	= ,,	2204
Net section modulus Z of two covers = I $\div$ 17·875 = 2204 $\div$ 17·875	= in.^{3}	124
Extreme fibre stress, tension = B.M. $\div$ Z = 753 $\div$ 124	= tons/sq. in.	6·08
Permissible stress in tension at 17·875″ from the neutral axis was	,,	6·2
Vertical shear stress in covers = 40 tons $\div$ gross area of two covers, = $40^{T} \div 2 \times 35\frac{3}{4}'' \times \frac{7}{16}''$	= ,,	1·28
Permissible shear stress in covers as for main web	= ,,	5

The covers are therefore suitable, and are those adopted in the general drawing of the plate gantry girder of a later chapter.

Rivet Modulus.—The formula for the moment of resistance of the rivets, newly found, is the $\frac{M}{I} = \frac{f}{y}$ form slightly disguised.

Thus the B.M. of 753

$$= 3 \text{ rows } @ \frac{h}{a} \times 2\Sigma a^2 + b^2 + c^2 + \ldots g^2$$

$$= \frac{h}{a} \Sigma 6a^2 + 6b^2 + \ldots 6g^2$$

$$= \frac{h}{a} \Sigma \text{ distances}^2 \qquad = \frac{h\,\Sigma \text{ distances}^2}{\text{extreme distance}}$$

Now h, the total horizontal load on the rivet, = rivet area A $\times f$, the stress per square inch, where A = the shear area (single or double) or the bearing area (*i.e.*, diameter $\times$ plate thickness), and f is respectively the shear stress in tons per square inch (single or double) or the bearing stress. Hence

$$\frac{h\Sigma \text{ dist.}^2}{\text{extreme dist.}} = \frac{f \times A\Sigma \text{ dist.}^2}{\text{extreme dist.}} = \frac{f \times I}{y}, \text{ since } I = A\Sigma \text{ dist.}^2$$

But $$Z = \frac{I}{y} = \frac{A\Sigma \text{ dist.}^2}{\text{extreme dist.}} = \frac{A(3 \times 1376{\cdot}36)}{16{\cdot}5} = 250{\cdot}2A.$$

$\therefore$ The stress in the extreme rivet $= f = M \div Z = 753 \div 250{\cdot}2A$ $= \frac{3}{A}$ tons/sq. in.

The horizontal load on the extreme rivet = stress in tons/sq. in. $\times$ area in sq. in. $= \frac{3}{A} \times A =$ as before, 3 tons.

Since the shear is double, then A	$= 2\times 0{\cdot}7854\times(\frac{15}{16})^2 =$	sq. in.	1·38
Whence the horizontal shear stress	$= 3^T \div 1{\cdot}38$ sq. in. $=$	tons/sq. in.	2·17
Similarly, the bearing area on the $\frac{3}{8}''$ web	$= \frac{3}{8}'' \times \frac{15}{16}''$ $=$	sq. in.	0·351
While the horizontal bearing stress	$= 3^T \div A = 3^T \div 0{\cdot}351$ $=$	tons/sq. in.	8·54
The vertical shear stress	$= 1^T \div 1{\cdot}38$ sq. in. $=$	,,	0·725
The vertical bearing stress	$= 1^T \div 0{\cdot}351$,, $=$	,,	2·85
The resulting bearing stress	$= \sqrt{[8{\cdot}54^2 + 2{\cdot}85^2]} =$	,,	9·0
The permissible bearing stress at the C. of G. of the flanges	$= 12$	,,	
and the permissible bearing stress at extreme rivet $= 12 \times$ ratio of the distances from N.A. $= 12 \times 16{\cdot}5 \div 23$	$=$	,,	8·61
The overload $= 9 - 8{\cdot}61$ in tons per square inch, or, as before,			4%

CHAPTER VIII

ECCENTRIC RIVETED CONNECTIONS

With the exception of the web splice carrying bending moment, all the previous examples of riveted joints carried, or were assumed to carry, direct load only. Cases arise where the rivets have to counteract bending moment in addition to direct load, as happened in the last examples of the web splice. If the bending moment is small it is neglected and only the direct load is considered. The simple lap joint is a case in point.

The resultant pull runs along the centre of gravity of each plate (Fig. 112), and at the joint these lines of force are at a distance e apart. The couple of P.e acts upon the joint and tends to bend the

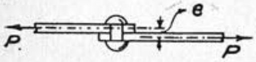


FIG. 112.

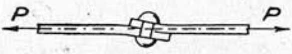


FIG. 113.

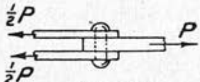


FIG. 114.

plates so that the two centre of gravity lines become collinear, as in Fig. 113.

The rivet shanks are now also in tension, as there is a lever action trying to force the rivet heads apart. Also, the thicker the plates the greater is the eccentricity e, and therefore the greater the moment of the couple. However, the rivet diameter increases with the plate thickness, so that, relatively, the bending stresses due to eccentricity are approximately the same on small-diameter rivets as on large-diameter rivets.

If this bending moment P.e, due to the couple, is to be neglected, as it is in practice, then there is no necessity for a great refinement of calculation. Further, when it is known that such a bending moment occurs it is really false economy to give the exact and minimum number of rivets required to carry the direct pull P in single shear or in bearing.

The double-butt strap joint has no such bending action provided the plates take their proper share of the load. This in itself is open to question if one desires to cavil over small things. The rivets may not fill the holes completely, or the plates may not all have the same elastic properties, and thus it is quite probable that a greater proportion than $\frac{P}{2}$ may be flung on to one or other of the two outer plates of Fig. 114.

Fig. 115 is as near the ideal connection as can be attained with lapped plates. The only eccentricity is the unavoidable and neglegible one due to plate thicknesses, Fig. *b*. The load on each of the four rivets is $\frac{1}{2}$ P.

In Fig. 116 the line of pull is no longer coincident with the axis of symmetry of the upper pair of rivets, whose centre of gravity lies at X. Assume the uppermost member rigid and a pin driven in at X replacing the two upper rivets, then the vertical flat and gusset would swing, pendulum fashion, as indicated by the large arrow.

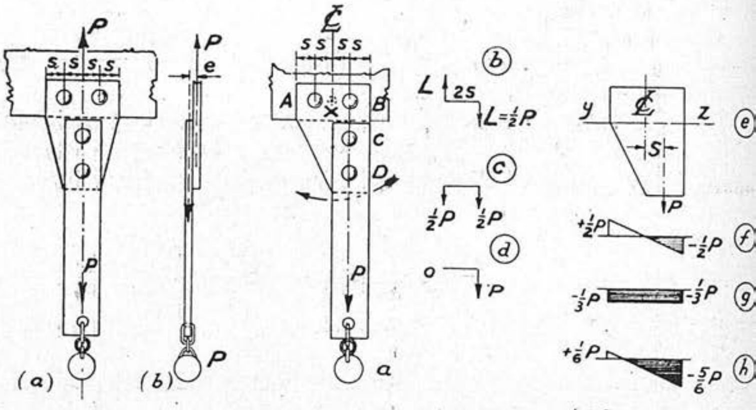


FIG. 115. FIG. 116.

Rivet A, therefore, is pushed upwards, while rivet B is pulled downwards, both with a force of, say, L tons. In other words, there is a clockwise couple acting on the rivets equal to L × distance AB = 2SL, Fig. *b*. The cause of this couple is the B.M. due to the eccentricity of P with respect to point X, the centre of gravity of rivets A and B. Therefore, 2SL = bending moment = P × S, hence L = $\frac{1}{2}$ P. In addition to this B.M. the upper rivets carry the direct load P equally between them, as indicated in Fig. *c*.

The resultant load on each of the upper pair of rivets is the summation of the loads of Figs. *b* and *c*, and is given in Fig. *d*. The latter figure shows that rivet B carries all the load of P tons, whereas the load on rivet A is nothing. Rivet B may be seriously overstressed, carrying, as it does, twice the load of either rivet C or D.

Similarly the centre line of the gusset plate does not coincide with the line of action of the external load P, so that the plate has to withstand a direct load P + a B.M. of P.S, Fig. *e*. The resulting stresses at any section *yz*, where the centre line is CL, are found as follows. Let the rivet pitch $= 2S = 3''$ and let the gusset plate thickness be $\frac{1}{2}$ in.

Cross-sectional area of gusset pl. at line *yz* $= 6'' \times \frac{1}{2}''$ $= 3$ sq. in.
Direct tensile load per sq. in. on line *yz* $= P \div 3 =$ (Fig. *g*), $-\frac{1}{3}P$.
The B.M. on the gusset is P × eccentricity S $=$ $1\frac{1}{2}$ P in. tons.
Section modulus along line *yz*, $(\frac{1}{6}BD^2)$ $= \frac{1}{6} \times \frac{1}{2} \times 6^2$ $= 3$ in.^{3}
Extreme fibre stress = B.M. ÷ section modulus $= 1\frac{1}{2}P \div 3$, (Fig. *f*) $= \pm \frac{1}{2}P$ tons/sq. in.
(Right edge in tension, left edge in compression.)

Adding the direct and bending stresses together (Fig. *h*), it is found that the left edge is stressed to $-\frac{1}{3}P + \frac{1}{2}P = +\frac{1}{6}P$ tons per square

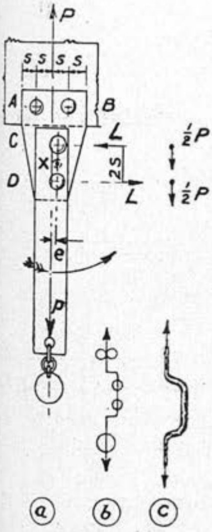


FIG. 117.

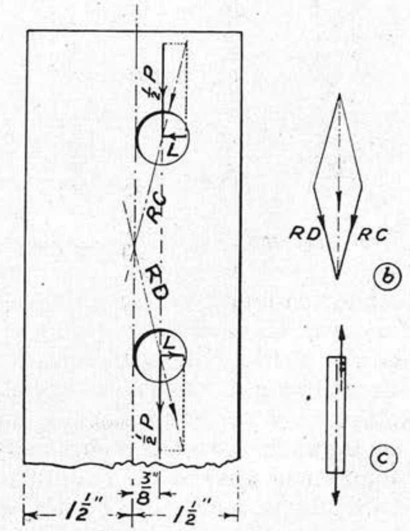


FIG. 118. ⓐ

inch, and the right edge is stressed up to $-\frac{1}{3}P - \frac{1}{2}P$, or $-\frac{5}{6}P$ tons per square inch, where + is the compression sign. As a check upon the calculations the average of these two fibre stresses should be equal to the direct load intensity, viz. $-\frac{1}{3}P = (-\frac{5}{6}P + \frac{1}{6}P) \div 2$.

Fig. 117.—The rivets in the flat bar are now eccentric. The

upper rivets in the gusset plate each carry a load, in tons, of $\frac{1}{2}$P, direct load only.

The flat bar rivets are not coincident with the line of pull of P, and are thus subjected to both a direct and a bending stress. The direct load for either rivet C or D is $\frac{1}{2}$P tons. The bending moment on these rivets is $P \times e$, where e is the eccentricity of the line of pull from the gravity centre X of the rivets. The counteracting couple of the rivets is $L \times 2S$, where L is the force per rivet and 2S is the pitch. In order to find the sense of L, assume both gusset and upper member rigid, and that a pin at X replaces rivets C and D. The flat will swing counter-clockwise about X in order that the weight may be directly under the pin.

As before, $P.e = 2LS$, or $L = P.e \div 2S$. The total load on either rivet C or D is found by adding, as vector quantities, $\frac{1}{2}$P to L. As a numerical example let P = 5 tons, $e = \frac{3}{8}$ in. and rivet pitch 2S = 3 in.

B.M.	$= P.e$	$= 5^T \times \frac{3}{8}''$	=	in. tons	$1\frac{7}{8}$
Couple	$= 2LS$	$= 2L \times 1\frac{1}{2}''$	=	,,	3L

Since $3L = 1\frac{7}{8}$ $\therefore$ L is $\frac{5}{8}$ ton.

The direct load per rivet is $5^T \div 2$ = tons $2\frac{1}{2}$

The resultant load on either C or D $= \sqrt{2{\cdot}5^2 + \frac{5}{8}^2} = 2{\cdot}58$ tons, Fig. 118.

Fig. 118.—There is no bending on the vertical bar of Fig. 117, since the line of pull is coincident with the centre of gravity line. This statement seems to confuse beginners as will be appreciated from Fig. 118*c*, which shows the vertical bar together with the student's usual conception of the forces acting on it. The forces, however, are incompletely shown because force L has been left out at the rivets. The proof that there is no bending on the vertical bar is obtained at once by finding the resultant of forces RD and RC, Fig. *b*. This resultant is found to be 5 tons exactly, acting on the flat's centre line, Fig. *a*, *i.e.*, the resultant is exactly equal, but opposite in sense to P.

Similarly there is no bending on the gusset, although at first sight this case appears to be similar to that of Fig. 116*e*. The downward force on the gusset is certainly applied off centre through the rivet holes by the rivets C and D, but the horizontal forces L bring the actual load line back on to the axis of symmetry of the gusset.

The analogy to the cranked tie-bar is illustrated by diagrams *b* and *c* of Fig. 117.

Fig. 119.—Here the pull is central with the gusset rivets and coincident with the flat bar rivets. All four rivets will carry,

therefore, a direct load only of $\frac{1}{2}$P tons per rivet. The flat bar is eccentric from the line of pull by an amount $e = \frac{3}{8}''$, and it is the bar, in this case, which is subjected to direct and bending stresses. Due to bending, caused by this eccentricity, the left-hand edge of the flat tends to compress while the right-hand edge tends to elongate. This follows at once from the pin test; *i.e.*, by imagining pin X placed on the centre of gravity line of the structural element

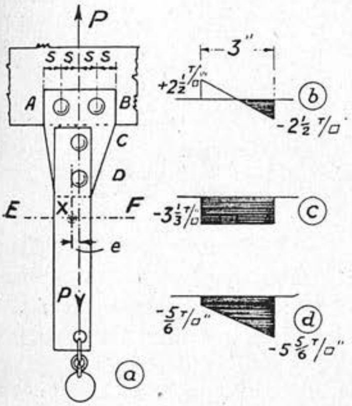


FIG. 119.

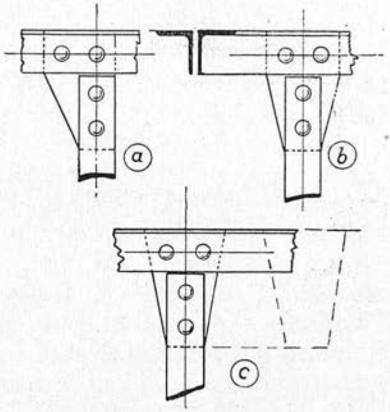


FIG 120

under consideration and then sketching the path of rotation of the element about this centre.

Assume the bar to be a $3'' \times \frac{1}{2}''$ flat, to find the stress distribution across any section such as EF.

Cross-sectional area at EF $= 3'' \times \frac{1}{2}''$ = sq. in. $1\frac{1}{2}$

Stress due to direct load, Fig. *c*, = P $\div$ area
$= -5$ tons $\div 1\frac{1}{2}$ = tons/sq. in. $-3\frac{1}{3}$

Bending moment on the bar $= \text{P} \times e =$
$5^{\text{T}} \times \frac{3}{8}''$ = in. tons $1\frac{7}{8}$

Section modulus Z of bar $= (\frac{1}{6}\text{BD}^2) =$
$\frac{1}{6} \times \frac{1}{2} \times 3^2$ = in.^{3} $\frac{3}{4}$

Extreme fibre stress due to B.M., Fig. $b =$
B.M. $\div$ Z $= 1\frac{7}{8} \div \frac{3}{4}$ = tons/sq. in. $\pm 2\frac{1}{2}$

Total extreme fibre stress, Fig. $d =$
$-3\frac{1}{3} \pm 2\frac{1}{2}$ = ,, $-5\frac{5}{6}$ & $-\frac{5}{6}$

As a verification the average of the two last stresses should give the direct stress, *i.e.* $6\frac{2}{3}^{\text{T}} \div 2 = 3\frac{1}{3}^{\text{T}}$.

The stress in the bar, when central with the load line, is $3\frac{1}{3}$ tons per square inch, but by giving the eccentricity of $\frac{3}{8}$ in. this stress has almost been doubled in the case of the extreme right-hand fibre.

Fig. 120 illustrates common roof truss details. Case *a* is bad, since the top right-hand gusset rivet is seriously overloaded, as proved in regard to Fig. 116. No defence can be offered for its continual use, not even on the plea of economy. It requires five cuts to make this gusset.

Detail *b* is much superior, as it has all the rivets centrally loaded. The gusset, however, requires six cuts.

Fig. *c* is the ideal connection for this case because the rivets are centrally loaded while the gusset plate has only four cuts. The gusset can be cut from one strip (see dotted lines) with practically no waste, only two end-pieces being left over. Compare this with the scrap entailed in making gusset plate *a*, where every gusset has a small triangular waste piece.

Fig. *b* appeals to some designers more than Fig *c*, but once the gusset is riveted in between the main rafter angles they both look the same in the finished roof truss.

CRANE BRACKET. *Figs.* 121 *to* 124.—The worst case of loading occurs when the crane and its load are drawn up close to the column. This gives the maximum vertical load. The lateral thrust due to the cross travel of the crab can be neglected, because the crab, if carrying full load, must go very slowly when in such close proximity to the end of its travel.

The question now arises as to how much load comes on to each bracket plate. If each bracket plate was independent of its neighbour, then either plate would, in turn, have to support the entire reaction due to the crane and its load. By using a diaphragm one side plate cannot deflect without taking its neighbour with it, and its neighbour cannot deflect unless it also participates in the carrying of the load, since deflection is proportional to load. There is another factor which is sometimes considered, and that is the frequency of this particular case of loading. The crane wheels will be often in the position shown, but it will be seldom that they will be called upon to carry the maximum load with the crab drawn in tightly against the bracket. The fact that maximum load is practically synonymous with maximum bulk explains this. The gantry girder was designed for the worst possible cases of loading without consideration as to the laws of chance, and to be consistent the vertical brackets should be designed to meet the maximum load. Only the question of the diaphragm is now left. Conservative designers make each bracket plate strong enough to carry all the reaction. Others, in competitive work, often assume that the unloaded plate relieves the loaded plate of a certain amount of load, the maximum load on the loaded plate being variously estimated at one-half, three-quarters and seven-eighths of the maximum girder reaction. The half is decidedly risky in view of the shallow depth of diaphragm and the resulting small landing left for pitching rivets therein, because the rivets through the plate into the diaphragm must, at the very least, be sufficient in number to carry that portion of the load which is assumed transferable to the idle plate.

In the following example the course chosen is neither too conservative nor too venturesome, as the active plate will be assumed to carry seven-eighths of the live load reaction, care being exercised that the diaphragm is of the requisite depth for rivet spacing.

Numerical Example.—Design a crane bracket for the 20 ft. span joist and channel gantry girder of Chapter XI., but attached to a slightly different type of column.

Dead load reaction from girder, at each bracket plate (item 2, girder calculations)	= tons	0·4
Live load reaction plus impact at one bracket plate (item 2, girder calculations) = 15·5 tons		
Assume live load on bracket plate as being $\frac{7}{8}$ of $15{\cdot}5^{\text{T}}$	= ,,	13·6
Total load P, Fig. 121, on one plate = $0{\cdot}4^{\text{T}} + 13{\cdot}6^{\text{T}}$ =	,,	14·0
Load carried to "idle" plate by diaphragm = $15{\cdot}5^{\text{T}} - 13{\cdot}6^{\text{T}}$ = say	,,	2
Permissible tensile stress per square inch, see girder calculations	= ,,	8
Max. B.M. on rivet group about the centre of gravity $X = P \times e = 14^{\text{T}} \times 13\frac{1}{2}''$	= in. tons	189

The group of rivets, therefore, carries a direct vertical load of 14^{T} plus a bending moment of 189 in. tons. The direct vertical load is equally shared by all the rivets and hence vertical load per rivet is

$$14^{\text{T}} \div 14 \text{ rivets} = 1^{\text{T}}.$$

If the plate rotated under the B.M. it would push that portion of rivet A which projects out of the column upwards and towards the right, the thrust being at right angles to the radial line XA, whose length is a. This thrust L would have the same numerical value at Q, but would be downwards and towards the left, again acting at right angles to the radial line XQ of length $q = a$. Now the nearer the neutral axis or centre of gravity the less is the thrust on the rivets, hence the load on rivet B will be $L \times \frac{b}{a}$ and on rivet C it is $L \times \frac{c}{a}$, as explained in the previous chapter.

A table similar to that employed in the calculations for the web splice carrying shear and bending moment of the preceding chapter can now be drawn up.

Rivet.	Load.	Radial Lever Arm.	Resisting Polar Moment = Load × Lever Arm.	
A	L	a	La or	L$a^2 \div a$
B	L $\times \dfrac{b}{a}$	b		L$b^2 \div a$
C	L $\times \dfrac{c}{a}$	c		L$c^2 \div a$

etc.

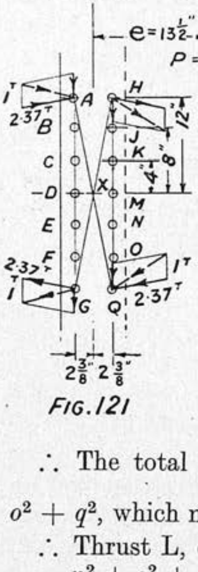


FIG. 121

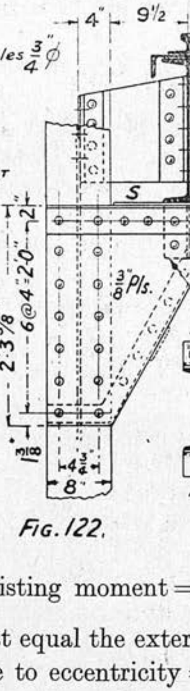


FIG. 122.

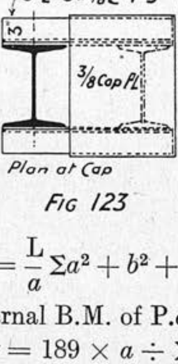


FIG 123

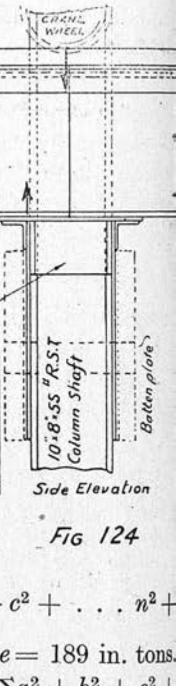


FIG 124

$\therefore$ The total resisting moment $= \dfrac{\mathrm{L}}{a}\Sigma a^2 + b^2 + c^2 + \ldots n^2 +$ $o^2 + q^2$, which must equal the external B.M. of P.$e =$ 189 in. tons.

$\therefore$ Thrust L, due to eccentricity $= 189 \times a \div \Sigma a^2 + b^2 + c^2 + \ldots n^2 + o^2 + q^2$.

By Euclid I — 47 : square on hypotenuse = sum of squares on other two sides. $\therefore a^2 = 12^2 + 2\frac{3}{8}{}^2$, (Fig. 121), $b^2 = 8^2 + 2\frac{3}{8}{}^2$ and $c^2 = 4^2 + 2\frac{3}{8}{}^2$.

Whence $\mathrm{L} = 189 \times 12{\cdot}23 \div [4(12^2 + 8^2 + 4^2) + 14(2\tfrac{3}{8})^2]$
$= 189 \times 12{\cdot}23 \div 975$ = in. tons 2·37

The total load on any rivet is found by drawing the parallelogram of forces for that rivet. Note that the arrow-heads of the forces must either be both away from the rivet or both towards the rivet.

Total load on the rivet A or G is the resultant of 1^T and $2{\cdot}37^T = 2{\cdot}38^T$.
Total load on the rivet H or Q is the resultant of 1^T and $2{\cdot}37^T = 2{\cdot}74^T$.

Note that the angle between the 1^T and $2{\cdot}37^T$ loads is acute for rivets H and Q and obtuse for rivets A and G.

From Table 12, Vol. III., the value per $\frac{3}{4}$ in. diameter rivet is $2{\cdot}65^T$, S.S. and $3{\cdot}38^T$ for $\frac{3}{8}$ in. bearing.

There are only two rivets, H and Q, which are slightly overstressed ($2{\cdot}74^T - 2{\cdot}65^T = 0{\cdot}09^T$ or 3 per cent.); all the remaining 12 rivets are under the permissible load of 2·65 tons per rivet. In view of the fact that the exact distribution of the load to the bracket plate is a matter of conjecture the riveting may be accepted as being quite satisfactory.

With the wheels as shown in Fig. 124, part of the load finds its way into the left side bracket plate through the diaphragm, and the remainder through the horizontal $3\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$ angle under the loaded side of the cap plate of Fig. 123. The two rivets at T are thus in double shear and $\frac{3}{8}$-in. bearing, while the two at S and the two at U are in single shear and $\frac{3}{8}$-in. bearing. The extreme left-hand rivet at S, however, can transfer but little load into the side plate.

The value of the riveting is therefore 2 @ $3{\cdot}38^T$, *i.e.*, $\frac{3}{8}$-in. bearing at T + 4 @ $2{\cdot}65^T$, *i.e.*, S.S. at S and U = $17{\cdot}36^T$, which is in excess of the load P of 14^T carried by these rivets.

Plate Calculations.

Gross moment of inertia of side plate at line HQ = " $\frac{1}{12}BD^3$ " $= \frac{1}{12} \times \frac{3}{8} \times 27\frac{3}{8}^3$	= in.^{4}	641·6
M. of I. of the holes about the N.A. = Σ area $\times$ dist.$^2 = (\frac{3}{8} \times \frac{3}{4}) \times 2(12^2 + 8^2 + 4^2)$	= ,,	126·0
$\therefore$ Net M. of I. of plate along line HQ = difference	= ,,	515·6
$\therefore$ Net section modulus Z = " $I \div y$ " = $515{\cdot}6 \div \frac{1}{2}$ of $27\frac{3}{8}$	= in.^{3}	38
The B.M. at HQ line = $14^T(13\frac{1}{2}'' - 2\frac{3}{8}'')$	= in. tons	155·8
Extreme fibre stress calculating with the net Z = M ÷ Z = 155·8 ÷ 38	= tons/sq. in.	$\pm$ 4·1

The B.M. at the HQ line is the maximum on the plate and, following a straight line law, the B.M. decreases gradually to zero as the plate is traversed towards the load line of P.

The plate is also safe against shearing along the vertical line HQ. Shearing force = P = tons 14

$\therefore$ Shear stress = $14^{T} \div$ gross area of $27\frac{3}{8}'' \times \frac{3}{8}''$ thick = tons/sq. in. 1·36

The shear stress should also be calculated for intermediate vertical sections between HQ and the load line of P.

Sometimes an edge angle ($2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ in this case) is riveted to the plate's compression edge so as to stiffen it against buckling. The maximum pitch of the connecting rivets must not exceed 12t. A batten plate is frequently riveted across the faces of the two edge

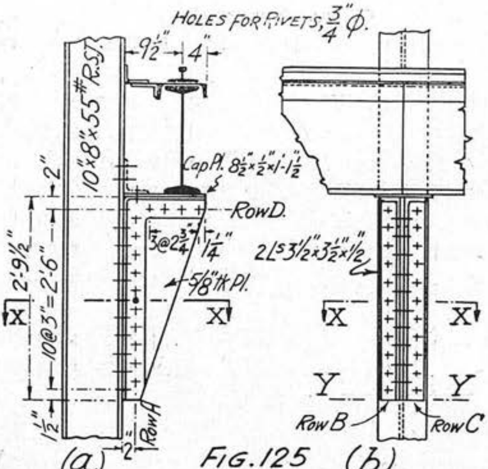
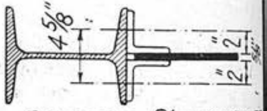
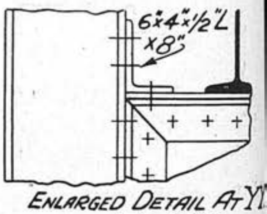


FIG. 125



ENLARGED PLAN AT X X.
FIG. 126.



ENLARGED DETAIL AT Y Y.
FIG. 127.

angles, Figs. 122 and 124, as a further precaution against buckling by lowering the $\frac{l}{r}$ value of the angle. The edge and cap angles if well riveted may be included when calculating the section modulus of the bracket side plate.

Another Numerical Example.—Figs. 125 to 127 show a type of bracket sometimes used when the web of the column shaft is at right angles to the crane girder. The upper rivets of the rows marked B and C are in axial tension, and to mitigate this evil they are often replaced by bolts (see p. 56), or the detail is amended after the manner of Fig. 128.

The thickness of the bracket plate is fixed by three things:—(1) The rivet values at D transferring the load from the cap into the vertical plate. (2) Section modulus for bending. (3) The vertical shearing stress on the plate.

Data as for previous example.

Rivet values, $\frac{3}{4}$-in. diameter holes. S.S. $= 2{\cdot}65^{T}$, D.S. $= 5{\cdot}30^{T}$ and $\frac{5}{8}$-in. bearing $= 5{\cdot}62^{T}$ per rivet, Table 12.

Load on bracket = dead load reaction from two girders + live load and impact $= 2 \times 0{\cdot}4^{T} + 15{\cdot}5^{T} = 16{\cdot}3^{T}$.

The weight of the bracket itself, about $1\frac{1}{4}$ cwt., is neglected.

Riveting at Row D.—Cap to plate.—The three rivets immediately under the girder are in D.S. and $\frac{5}{8}''$ B $= 3 \times 5{\cdot}30^{T} = 15{\cdot}90.^{T}$ This falls short of $16{\cdot}3^{T}$ by $0{\cdot}40^{T}$, which may be assumed to be carried by the remaining rivet at the extreme left of row D. These rivets are considered to be centrally loaded.

Vertical Plate.

The B.M. at the rivet line A	$= 16{\cdot}3^{T} \times 7\frac{1}{2}''$	= in. tons	122·25
Gross M. of I., at line A, of vertical plate	$= \frac{1}{12} \times \frac{5}{8}'' \times (33{\cdot}5'')^3$	= in.⁴	1,958
M. of I. of the holes on line A = Σ area × dist.² $= \frac{5}{8}'' \times \frac{3}{4}'' \times 2[1{\cdot}5^2 + 4{\cdot}5^2 + 7{\cdot}5^2 + 10{\cdot}5^2 + 13{\cdot}5^2]$		= ,,	348
∴ Net M. of I. (Neglecting M. of I. of holes about their own axis—extremely small)		= ,,	1,610
Net section modulus Z of plate (neglecting horizontal cap angle) $= 1{,}610 \div \frac{1}{2}$ of $33{\cdot}5$		= in.³	96
Extreme fibre stress on plate, based on net Z $= 122{\cdot}25 \div 96$		= T/sq. in.	±1·3
The vertical shearing stress per gross sq. in. $= 16{\cdot}3^{T} \div$ gross area of $33\frac{1}{2}'' \times \frac{5}{8}''$		= ,,	0·78

Riveting at Row A.—The vertical plate does not butt hard against the column face, but is kept $\frac{1}{4}$ in. back, as in Fig. 126, because of the sheared edge, and, therefore, the centres of gravity and rotation of the rivets in row A are coincident with the dot on the XX line of Fig. 125*a*. The load L, due to bending, on the top and bottom rivets is

$$\frac{\text{B.M.} \times \text{extreme rivet distance}}{\Sigma\ \text{dist.}^2}, \text{ as in previous example,}$$

$$= \frac{122{\cdot}25 \times 13{\cdot}5}{2[1{\cdot}5^2 + 4{\cdot}5^2 + 7{\cdot}5^2 + 10{\cdot}5^2 + 13{\cdot}5^2]} = \text{tons} \qquad 2{\cdot}22$$

The load on the next outer rivet is $2{\cdot}22 \times 10{\cdot}5 \div 13{\cdot}5$. These thrusts all act horizontally, *i.e.*, at right angles to the vertical radial line.

The direct load per rivet is $16{\cdot}3^{T} \div 10$, acting vertically downwards		= tons	1·63
Resulting load on outermost rivets	$= \sqrt{[1{\cdot}63^2 + 2{\cdot}22^2]} =$	,,	2·75
Permissible load on outermost rivets	= D.S. value per rivet of	,,	5·30

The A row of rivets is amply safe, but the number is fixed by the number of B and C rivets required, with which rows the A rivets are reel-pitched.

Riveting at Rows B and C.—As in the case of the A rivets take the distances from the XX line, the centre of gravity line of the group—a further discussion on this point is given later in connection with Figs. 129 to 132.

$$\text{B.M. at column face} = 16{\cdot}3^{T} \times 9\tfrac{1}{2}'' = \text{in. tons} \qquad 154{\cdot}85$$

The upper rivets are in tension while the lower ones are relieved of axial load, since the bracket angles are compressed against the column face.

Axial tensile load on topmost pair of rivets

$$= \frac{\text{B.M.} \times \text{extreme radial length}}{\Sigma\ \text{distance}^2}$$

$$= \frac{154{\cdot}85 \times 15}{4\Sigma 3^2 + 6^2 + 9^2 + 12^2 + 15^2} = \text{tons } 1{\cdot}17$$

Direct vertical shearing load on each of the B and C rivets $= 16{\cdot}3^{T} \div 22$ rivets	= ,,	0·74
∴ Axial tensile stress on the topmost pair of rivets, per sq. in., $f_t = 1{\cdot}17 \div 0{\cdot}44$ sq. in.	= ,,	2·66
Direct shearing stress on each of the B and C rivets, per sq. in., $f_s = 0{\cdot}74 \div 0{\cdot}44$ sq. in.	= ,,	1·68

From the theory of principal stresses (see text-books on Strength of Materials) the maximum shear stress v and the maximum tensile stress f on the uppermost pair of rivets are :—

$$v = \sqrt{\left(\frac{f_t}{2}\right)^2 + (f_s)^2} = \sqrt{\left(\frac{2{\cdot}66}{2}\right)^2 + 1{\cdot}68^2} = \text{tons/sq. in.} \quad 2{\cdot}14$$

$$f = \frac{f_t}{2} + \sqrt{\left(\frac{f_t}{2}\right)^2 + (f_s)^2} = \tfrac{1}{2} f_t + v$$

$$= \tfrac{1}{2} \text{ of } 2{\cdot}66 + 2{\cdot}14 = \quad ,, \quad ,, \quad 3{\cdot}47$$

The standard centre of holes in the joist flange of $4\frac{3}{4}$ in. has been altered to $4\frac{5}{8}$ in. to suit the holing of the angles. The reverse shelf angle, $6'' \times 4'' \times \frac{1}{2}''$, of Fig. 127, may be used in detail (Fig. 125*a*) as dotted, since it adds considerably to the bracket strength (the resisting moment of its rivets can be taken into account, but this is not usually done).

Fig. 127.—The bracket angles of Fig. 125*a* are cut, kneed and smith welded at the bend. The efficiency of this type of weld is very variable and its maximum value is in the region of 70 per cent. Many designers eliminate this weld and substitute for it the mitred corner of Fig. 127, in conjunction with the $6'' \times 4'' \times \frac{1}{2}''$ angle cleat. The expenditure of 11 lb. of metal in this cleat is more than paid for in the saving of the weld. From the remarks in the succeeding article it follows that the vertical leg of the $6'' \times 4'' \times \frac{1}{2}''$ angle cleat should have more rivets in it than are in the horizontal leg, the former rivets being in tension and the latter in single shear.

The Permissible Axial Tensile Stress on Rivets is difficult to ascertain due to the high and variable initial tensile stresses set up when the newly-clenched rivet cools, as was emphasised in Chapter IV. It is certainly not good practice to have rivets in tension, but cases do arise in practice where rivets in tension can hardly be avoided. By using a low working stress for these rivets and subjecting them to rigid inspection as to soundness and fullness of head, quite efficient and trustworthy joints can be obtained. It is suggested here that the permissible tensile load on a rivet be taken as one-half of the single shear value of the same rivet. Thus, for a $\frac{3}{4}$-in. diameter rivet the S.S. value is $2{\cdot}65^{\text{T}}$, hence the permissible axial tensile load may be taken as $\frac{1}{2}$ of $2{\cdot}65^{\text{T}} = 1{\cdot}325^{\text{T}}$ per rivet, which is equivalent to 3^{T} per square inch of rivet steel when the f_t for the plate and section steel is 8^{T} per square inch. The writer's suggested rule can now be restated thus:—Take the axial tensile stress for rivet steel as three-eighths of the permissible tensile stress for plate and section steel. The minimum ultimate tensile strengths for plate steel and rivet steel are, respectively, 28 and 25 tons per net square inch. The "factor of safety," in the present instance, for the rivet steel is at least $25 \div 3 = 8\frac{1}{3}$, which is not too high, since allowance has to be made for such contingencies as the cooling stresses, badly-formed rivets overlooked through improper inspection, etc.

To ensure an equitable distribution of the load among the tensile rivets in angles B and C, Fig. 125, it is further suggested that these bracket angles should have a minimum thickness equal to two-thirds the rivet diameter less $\frac{1}{16}$ in., *i.e.*, $\frac{2}{3}$ of $\frac{3}{4}'' - \frac{1}{16}'' = \frac{7}{16}$ in$''$ in the present example. When there is only one angle connecting the bracket plate to the column face the tensile loads on the rivets through the column flange are no longer axial, because of the eccentricity of the applied load, and the thickness of the angle should be at least two-thirds the rivet diameter plus $\frac{1}{16}$ in.

I 2

Permissible Axial Tensile Stress on Bolts for riveted steelwork varies with the designer, and greatly depends upon his knowledge and his temerity with regard to initial screwing up stresses. The tensile stress is calculated on the cross-sectional area at the bottom of the threads, and the following is offered as a tentative guide.

Bolts under $\frac{3}{4}$ in. diameter working stress on core area $= \frac{1}{2}f_t$
$\frac{3}{4}$ in. diameter up to $\frac{7}{8}$ in. diameter working stress on core area $= \frac{5}{8}f_t$
Over $\frac{7}{8}$ in. diameter and up to 1 in. diameter working stress on core area $= \frac{3}{4}f_t$
Over 1 in. and up to 2 in. diameter working stress on core area $= \frac{7}{8}f_t$
Bolts over 2 in. diameter working stress on core area $= f_t$

Where $f_t =$ permissible axial tensile stress, which may be varied to suit the nature of the load, or, alternatively, as was done in the data of the foregoing numerical examples, f_t may be kept constant at 8 tons per square inch, and the live load increased by a variable impact factor; this factor depending upon the dynamic effect of the load, etc.

If $\frac{3}{4}$-in. diameter bolts be substituted for the tensile rivets in the upper ends of rows B and C in Fig. 125, the working axial load would be, core area $\times \frac{5}{8} \times 8^{T} = 0{\cdot}3 \times \frac{5}{8} \times 8^{T} = 1{\cdot}5$ tons. It would be better to use $\frac{7}{8}$-in. diameter bolts, and so decrease the bracket depth.

Provided that the bolt is not screwed or threaded too far up the shank, the shear stress is calculated upon the gross or shank area and not on the area at the bottom of the threads.

Fig. 128 is a riveted bracket wherein an attempt is made to minimise the tensile loads on the top rivets. A bracket of this type can be made much shallower than that of Fig. 125.

Figs. 129 to 132.—In the first of these figures the bracket undoubtedly tends to heel about the bottom pair of bolts, and some text-books assume that the bracket of Fig. 125 does the same. When finding the resisting moment of rows B and C, they take the centre of rotation as being on the line YY at the bottom of the vertical angles, and the summation of the distances² as $2(1{\cdot}5^2 + 4{\cdot}5^2 + 7{\cdot}5^2 + \ldots + 28{\cdot}5^2 + 31{\cdot}5^2)$, *i.e.*, all the rivets in rows B and C are assumed to be subjected to axial tension. This method results in a very shallow bracket, and must seriously overstress the tensile fastenings. There is no reason for assuming the bracket to heel at the bottom. If there was an absolutely rigid projection from the column at the line YY, then something might be said for this method. When considering the plate girder web splice for bending, rotation was not assumed at the top or compression edge O (Fig. 131), but at the neutral axis NA. Similarly, with a cantilevered girder (Fig. 132), the web splice would not be calculated on the assumption that the cantilevered nose heeled about the compression edge at O, and, carrying the reasoning one step further, this is really the case of the bracket.

The centre of rotation is somewhere between the XX line and the YY line of Fig. 125. If it were actually at the bottom, then the intensity of thrust upon the angles must be very large at this particular point, so large, indeed, that the angles B and C would be bent outwards from the column and the bottom rivet of row A sheared, which never occurs. The method of calculaton adopted for Fig. 125

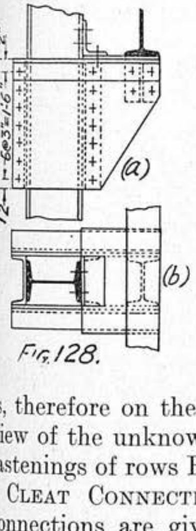


FIG. 128.

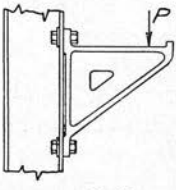


FIG. 129.

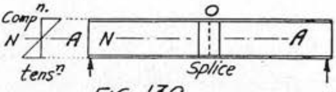


FIG. 130.

FIG. 131.

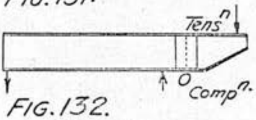


FIG. 132.

is, therefore on the safe side and should be adopted, especially in view of the unknown value of the initial tensile stresses in the top fastenings of rows B and C.

Cleat Connections.—Fig. **133**. Lists of "standard" cleat connections are given in the "section books" of various firms. These should not be slavishly copied as being the proper connection for the joists written against them, but rather should be looked upon as suggestions or guides and nothing more. The diagram under discussion is the proposed "standard" cleat for a $14'' \times 5\frac{1}{2}'' \times 40$ lb. R.S.J. On a 14-ft. span the uniformly distributed load necessary to cause an extreme fibre stress of 8 tons/square inch at mid-span is 20·5 tons. With this loading the end shear is 10·25 tons, while the shear stress on the web is $10{\cdot}25^{T} \div$ web area of $14'' \times 0{\cdot}37'' = 2^{T}$/sq. in. (B.S.S. (4/4)), against the permissible stress of 5^{T}. The same joist carrying a uniformly distributed load of 51·8 tons on a 5-ft. 6-in. span has the maximum transverse fibre stress at mid-span of 8^{T}/sq. in., while the end shear stress on the web is $25{\cdot}9^{T} \div$

$14'' \times 0{\cdot}37'' = 5^{T}$/sq. in., *i.e.*, both the permissible transverse and shear stresses are reached simultaneously. It would therefore be absurd to use the same cleat for a 14-in. R.S.J. for all spans and loading, as in the first example the end shear to be carried was $10{\cdot}25^{T}$, and in the second it was $25{\cdot}9^{T}$. In view of this the section book from which this "standard" was taken limits the span to not less than 14 ft.

Loads on the Rivets in the Web of the 14*-in. Joist.*—Span of joist 14 ft.; end reaction, as in the first example, $10{\cdot}25^{T}$.

The centre of gravity of the four rivets A to D is, by inspection, at X. Had the rivets been unevenly spaced, then the centre of gravity would be found by taking moments about a horizontal and a vertical line of reference.

Bending moment about $X = P.e = 10{\cdot}25^{T} \times 3\frac{3}{8}''$	= in. tons	34·6
Σ distances$^2 = XA^2 + XB^2$, etc. $= 4XA^2$ $= 4 \times 3{\cdot}204^2$	= in.^{2}	41·1
Load L on each rivet due to B.M. = ($P.e \times$ extreme radial length) $\div \Sigma$ dist.^{2} (and acts clockwise and at right angles to each radial line) $= 34{\cdot}6 \times 3{\cdot}204 \div 41{\cdot}1$	= tons	2·7
Direct vertical load per rivet $= P \div$ number of rivets $= 10{\cdot}25^{T} \div 4$	= ,,	2·56
Resulting load on the A and B rivets, acute angle between the forces	= ,,	4·35
Resulting load on the C and D rivets, obtuse angle between the forces	= ,,	2·99

The rivets are $\frac{3}{4}$ in. diameter and $f_t = 8^{T}$/sq. in., $\therefore$ D.S. value per rivet $= 5{\cdot}3^{T}$ and the bearing value on the 0·37-in. web is $3{\cdot}33^{T}$, from which it would appear that the A and B rivets are seriously overstressed to the extent of 31 per cent.

The above treatment is given in text-books without further remark, and it would appear at first sight that the quite common practice of designing cleat connections for the direct load only—*i.e.*, neglecting the bending moment of $P.e$—is greatly in error. However, there is one important assumption made in the foregoing calculations, viz., that the bolts E, F, G, H, I and J are in single shear only. On this basis the thrust from the beam causes a downward load on each bolt of $10{\cdot}25^{T} \div 6 = 1{\cdot}71^{T}$.

In practice the deflection of the 14-in. joist is small, and thus the slope of the joist at its ends must also be small, but, however small it may be, it will always create axial tension in the upper bolts E and H. These must elongate a trifle under stress, and so the cleat faces E and H also tend to rotate as well as to move vertically downwards.

Again, if E to J were rivets, and A to D were bolts with $\frac{1}{16}$ in. clearance in the holes and with the nuts not screwed too tightly up against the cleat faces (*i.e.*, loose pins), then the load on the four bolts A to D would be central through X, since the joist is free to deflect a small amount. The eccentricity of *e* still exists, but now causes a B.M. of P.*e* on the rivets E to J. This connection, however, is not used in practice, as it entails the threading of a heavy joist between two cleats.

Any simple practical calculation for strength must necessarily be based upon some questionable assumption. Thus the following have all a bearing upon the load distribution :—Faces not absolutely flat; clearance of bolts in holes; different degrees of tightness of

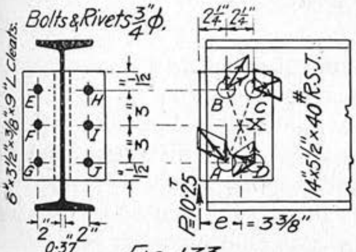


FIG. 133.

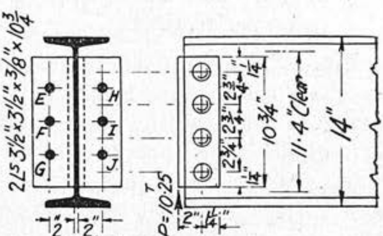


FIG. 134.

nuts and rivet heads against the cleat faces ; relative rigidity of the joist and the parent structure ; initial axial stress in the rivets and bolts ; and lastly, but not least, the frictional resistances of faces in contact.

Some attempt has been made to standardise the calculation of cleat connections, but it must be admitted that the great majority of designers when engaged upon beam work simply design the connection for direct load only, and this with no apparent ill effects.

Fig. 134.—With the knowledge that eccentricity stresses do exist a satisfactory cleated connection can be designed which will be quite in keeping with the stricter theoretical investigation given above by :—(1) Reducing the lever arm *e* as much as possible, and (2) giving the number of rivets slightly in excess of the direct load requirements. Table 13 (Vol. III.) states that the maximum clear depth between the root fillets of a 14-in. joist is 11·4 in., thus allowing the use of the 11-in. deep cleat. The eccentricity *e* can be reduced to 2 in. by using $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles.

had time, as it were, to permeate to the outermost fibre of the angles. Thus at the girder centre line, where the stress has reached its maximum intensity, the angles of the bottom flange are assumed to be evenly stressed throughout their cross-sectional area. The diagonals, on the other hand, had their loads given to them instantaneously by the end gussets.

Compression Members are assumed to be effective in carrying stress throughout their full cross-sectional area with no deductions for the outstanding leg. In the case of a strut the further the metal is away from the centre line the greater is the radius of gyration and the stronger is the member. A hollow shaft is stronger as a column than a solid shaft, where both columns have the same cross-sectional area of metal.

Tension members used to be made always of flat bars, but modern practice uses angles or built-up sections where possible. The angle gives a greater rigidity to the structure than the flat bar, and rigidity is of as great importance as strength. In many light bridges flat bar tension members can be seen to vibrate during the passage of the load, while the compression members, being necessarily of a section other than that of a flat bar, can hardly be observed to vibrate at all. Again, the girder may have to act either intentionally or unintentionally as part of a portal frame, and there is thus the possibility of a stress reversal in some members. Angles can take this reversal from a tensile to a compressive stress, whereas flat bars are unfitted to carry a compressive load due to their small radius of gyration in one direction.

FRAME CENTRE LINES.—The skeleton frame of the truss of Fig. 137, formed by the "centre lines," is shown on the left hand and the completed truss minus gusset plates is indicated on the right. The "centre lines" may be either the rivet lines of the angles or the centre of gravity lines. Since these two sets of lines are not coincident, there is always an eccentricity of loading either on the rivets or on the bars. Roughly the practice is :—(1) Where only angles are used the skeleton frame is composed of the rivet lines, and in the case of angles with double rivet lines the "centre lines" are the rivet lines which are nearer the heels.

(2) Where compound sections are used the "centre line" is the centre of gravity line of the built-up section. These centre lines are laid down on the paper first and the angles or compound sections placed in their relative positions afterwards.

Eccentric Frame Centre Lines.—It is axiomatic that the centre lines, whether gravity or rivet lines, should intersect at the panel points. This should be carried out in practice, although there are cases where this rule may be relaxed, particularly in light structures, but only after due regard has been paid to the stresses involved.

Fig. 138 is a common attempt to economise on gusset plates, which are absolutely necessary, as the correct detail of Fig. 139

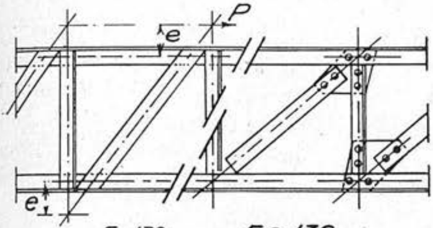


FIG. 138. FIG. 139.

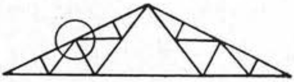


FIG. 140.

indicates. The bending moment P . e is shared by the members directly as their moments of inertia and inversely as their lengths. P is the increment of flange stress at the panel point, *i.e.*, the difference of the stresses in the portions of the flange to the left and right of the panel point. If I = moment of inertia and L = the length of a bar, then the proportion of the B.M. carried by a

$$\text{bar A} = \left(\frac{P\,.\,e}{\Sigma\frac{I_A}{L_A}+\frac{I_B}{L_B}+\frac{I_C}{L_C}\;\ldots}\right)\times\frac{I_A}{L_A}$$

The total stress in bar A is the direct or primary stress added to the stresses due to bending or secondary stresses.

Fig. 141 is the mid-rafter detail of Fig. 140 and has the simplest gusset plate possible. The number of rivets through the gusset

A = 2Ls 3"×2½"×¼" B = 2Ls 2½"×2½"×¼" C = 1L 2½"×2½"×¼" D. (Fig 144) 1L only.

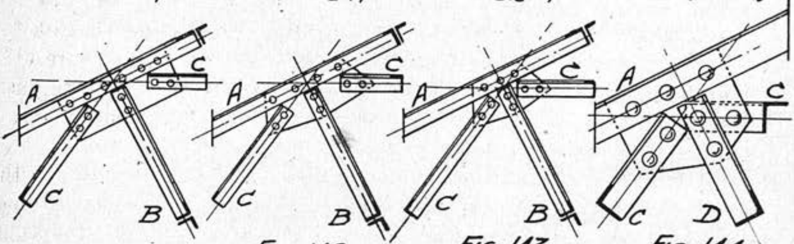


FIG. 141. FIG. 142. FIG. 143. FIG. 144.

plate and rafter angles A is generally small, and extra rivets are given for the purpose of stitching the elements together (12t, B.S.S. (4/26)).

Fig. 142 is an effort to lighten the appearance of the joint and, at the same time, to economise in weight of material and riveting, but the latter is only gained at the expense of the labour expended upon the plate. Too sharp a corner formed in the gusset plate where it overlies the bars C should be guarded against, as tapered corners spring away from the more rigid section. Both the present and the previous details satisfy the principle of intersection of centre lines.

Fig. 143.—The gusset plate is here reduced to a minimum consonant with symmetry, but only by introducing secondary stresses caused by the couple P . *e*, as in Fig. 138.

Fig. 144, drawn to twice the scale of its neighbours, exhibits still another fairly common attempt to cut down gusset area and

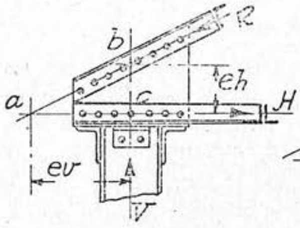


FIG. 145.

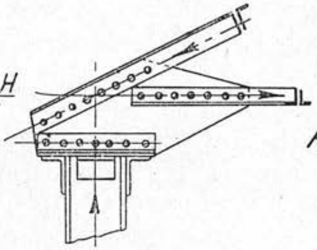


FIG. 146.

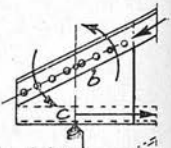


FIG. 147

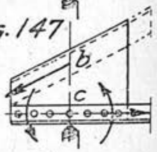


FIG. 148.

riveting. Frame centre lines are eccentric, and so also are the rivet groups. False economy with gusset plates may easily result in a dangerous structure.

Fig. 145 illustrates the common, almost universal, British detail for a roof truss shoe. The members have secondary stresses, because of the eccentricity of the centre lines, in addition to the primary stresses, while the rivets carry a direct load plus a load due to bending moment.

The bending moment $= V \cdot ev = H \cdot eh$, because $\frac{H}{R} = \frac{ac}{ab}$ and $\frac{V}{R} = \frac{cb}{ab}$, and so $V \cdot ev = R\frac{cb}{ab} \cdot ac$ and $H \cdot eh = R\frac{ac}{ab} \cdot cb$.

A clearer conception of the bending action can be obtained from Figs. 147 and 148. In the former the tie is replaced by its pull acting from the rivet centre of gravity *c*, and in the latter the rafter is assumed to be non-existent and replaced by its thrust acting at the rivet centre of gravity *b*. The gusset plate, which is cantilevered out from the remaining angles, transfers the counter-clockwise couple into those angles through the rivets. The bending moment is therefore shared by both rafter and main tie through their rivets, and, assuming a knife-edge support at the column cap under c, the bending moment on the rafter would be

$$\frac{I_R}{L_R} \times \frac{V \cdot ev}{\frac{I_R}{L_R} + \frac{I_T}{L_T}} :$$

the subscripts R and T denoting rafter and tie respectively. There is, however, a large counteracting moment offered by the rivets through the cap cleats which greatly reduces the bending moment on both rafter and tie.

Fig. 146.—This detail eliminates the foregoing secondary stresses, but is more used in America than here. It is not always advantageous, however, to use this detail. With wind bracing at roof tie level the wind load must get into the column shaft and so to the ground. In that detail more common to this country the shoe horizontal base plate is extended into the shop and also acts as a gusset plate for the horizontal wind-bracing system. The load is thereupon transferred into the column without passing through a raised and slender vertical plate. This point is reverted to in Chapter VI, Vol II.

If the vertical gusset plate of detail 145 be made $\frac{1}{16}$ in. or $\frac{1}{8}$ in. thicker than the other gusset plates of the truss and if the rivets be given slightly in excess, as previously advocated, then there is no objection to the use of this type of shoe, in fact this shoe is preferred by many as being more efficient and rigid.

REFERENCES

All the standard text-books upon the theory of structures devote more or less space to the theory of eccentric joints.

Institute of Structural Engineers. *Report on Steelwork for Buildings, Part II. Eccentrically Loaded Rivet Groups.* Price 1*s.* 6*d.*

Clarke, J. B. *The Practical Designing of Structural Steelwork Details. The Structural Engineer*, Vol II., No 1.

CHAPTER IX

WIND PRESSURE—FACTORS OF SAFETY

"The vagaries of the wind" is an expression used lightly by many, but not by those engaged upon the competitive design of high structures, to whom the term is pregnant with meaning. The wind is so erratic in behaviour that it is impossible to foretell the actual maximum wind pressure which will act upon a structure, and, since it flouts any attempt to be curbed by a formula, the designer has, perforce, to fall back on experience. Certainly a structure can be designed to withstand any wind which may blow, but economy demands that a structure need not withstand a tornado when there is no possibility of such an occurrence.

Pressure and Velocity have been connected together, however, by the formula $P = KV^2$, where P is in pounds per square foot of surface and V is the velocity of the wind in miles per hour. The constant K has undergone many vicissitudes, and the latest value assigned to it is 0·0031 for a flat plate placed normal to the wind, *i.e.*, $P = 0{\cdot}0031V^2$.

The Pressure-Velocity Constant K in $P = KV^2$ varies with the surface exposed to the wind. The following are some of the results of the Melbourne University experiments on wind pressures obtained in 1891—94. It will be noted that K for a horizontal wind pressure on a normal surface compares very favourably indeed with the value now established. With regard to wind pressures on oblique surfaces, Duchemin's formula, mentioned later, is most often employed.

Value of K in pressure-velocity formula . . . 0·0033

Relative pressure per square foot calculated for the cross-section of greatest area normal to the direction of the wind.

(1) Thin plate of square form	1·00
(2) Cube, one face normal to the wind . .	0·9
(3) Cube, one face diagonal to the wind . .	0·63
(4) Square tower, height three times base: as for (2) and (3).	
(5) Cylindrical tower	0·52
(6) Octagonal tower	0·6

Also see the B.S.S. (3/7) for the number of exposed elevations which should be taken in bridge design for wind stresses.

Velocity and Height above Ground.—The pioneering records of storm pressures at the Forth Bridge (Arrol's handbook) have now been found to be too high through errors due to inertia, nevertheless, the ratio existing between the readings will remain fairly constant although the absolute values may be somewhat incorrect. P is the

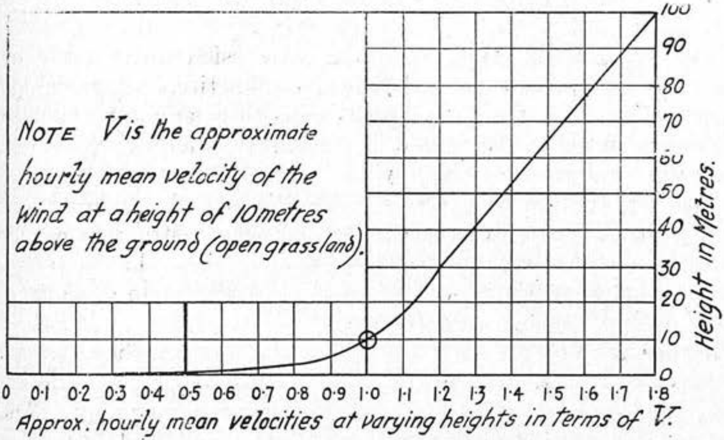


FIG. 149.

average pressure for a period of several years in pounds per square foot on the gauge at the lowest level.

Gauge No.	4	50 ft. above H.W.	Pressure	= P
,,	2	163 ft. ,,	,,	= 1·77P
,,	1	214 ft. ,,	,,	= 2·15P
,,	5	214 ft. ,,	,,	= 2·31P
,,	3	378 ft. ,,	,,	= 3·84P

These indicate that the pressure (and therefore the velocity) increases as the height of the instrument is increased.

Fig. 149 is a rough drawing of a chart published by the Meteorological Office (1918) in their *Monthly Weather Report*, which gives further evidence on this point. If the velocity of the wind at a height of 10 metres be called V, then the velocity at any other height is given by the graph in terms of this V.

Pressures on Oblique Planes.

Let P be the horizontal pressure in pounds per square foot of vertical surface, and P_n the resulting normal pressure, also in pounds per square foot, then :—

(1) $P_n = P \dfrac{2 \sin \theta}{1 + \sin^2 \theta}$ . . . . (Duchemin.)

(2) $P_n = P \dfrac{\theta^\circ}{45^\circ}$, where θ is not greater than 45°. (Straight line.)

(3) $P_n = P \sin \theta^{\,1\cdot84 \cos \theta - 1}$ . . . (Hutton.)

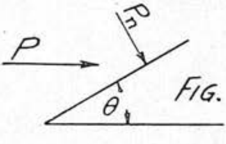
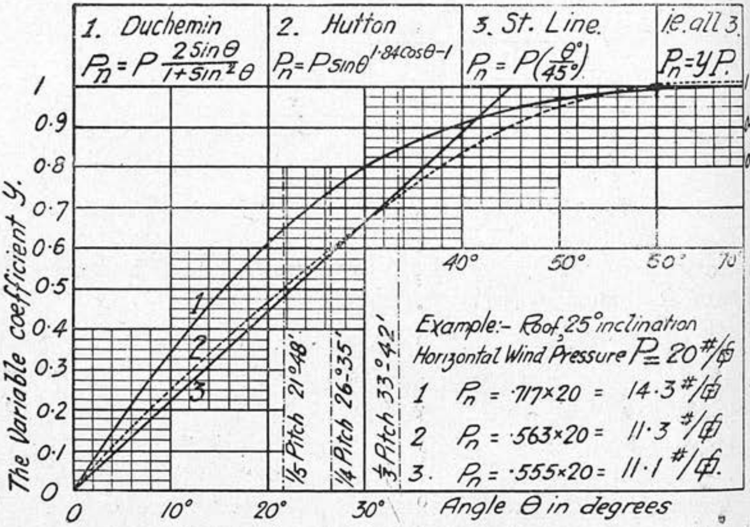


FIG. 150

Of the three formulæ Duchemin's is the most common, but the straight line is simpler and quite in keeping with our knowledge of wind pressures. Hutton's formula is too complicated and is therefore hardly justifiable in view of the known eccentricities of the wind. The three formulæ are graphed in Fig. 151.

Pressure and Locality.—The configuration of the ground and adjoining structures both influence the wind velocity and, therefore,



The values of the coefficients in the wind pressure formula

FIG. 151.

the pressure. A bridge which spans a deep funnel-shaped gorge may be expected to experience a higher wind pressure than one which bridges a placid stream in a grassy plain, while a low building entirely surrounded and screened by high warehouses may never be subjected to any wind pressure worth mentioning. Also see the last sentence of Article 7, Part 3, of the B.S.S.

Maximum Velocities in Great Britain.—Dr. Stanton remarks that only on two known occasions has the wind ever attained a velocity of 110 miles per hour in Great Britain and that both sites were exceptionally exposed ones.

Mr. Remfry finds on examination of the records of most of the important stations in Great Britain for the years 1906 to 1913 that the maximum velocity in gusts were 99 m.p.h. once, 90 m.p.h. twice, 85 to 89 m.p.h. nine times.

Admiral Beaufort (1805), in his famous but now obsolete scale for seamen, stated that a whole gale was one which was "seldom experienced inland; trees uprooted; considerable structural damage done." Considerable damage was caused in Edinburgh by the storm of January 28th, 1927, which was the worst registered

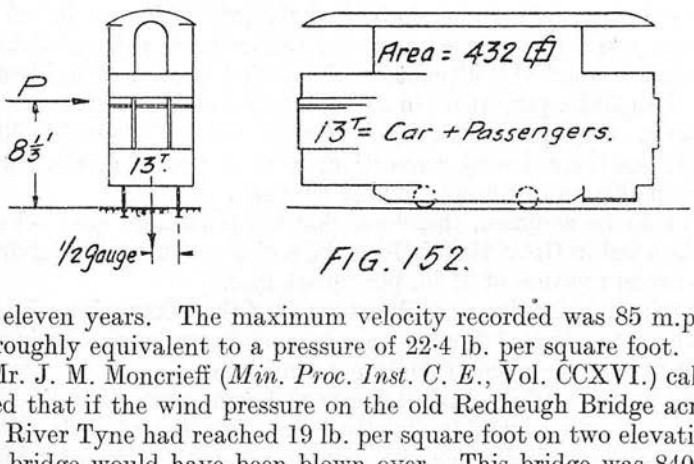


FIG. 152.

for eleven years. The maximum velocity recorded was 85 m.p.h., or roughly equivalent to a pressure of 22·4 lb. per square foot.

Mr. J. M. Moncrieff (*Min. Proc. Inst. C. E.*, Vol. CCXVI.) calculated that if the wind pressure on the old Redheugh Bridge across the River Tyne had reached 19 lb. per square foot on two elevations the bridge would have been blown over. This bridge was 840 ft. long with two central spans each of 252 ft., while the top of the main girder was 128 ft. above low water. Yet this bridge successfully withstood many violent storms for twenty-five years.

Again, because of the flaw in the main tie-rod of the Charing Cross Station roof, which eventually caused its collapse after forty years in 1905, it was calculated that the structure had never been subjected to a horizontal wind pressure higher than 27 lb. per square foot (*ibid.*, p. 43, Mr. Ellson).

Sir Benjamin Baker's experiments with a glazed window as the bottom of a water tank led him to believe that ordinary windows would break at a pressure somewhere in the neighbourhood of a pressure of 30 lb. per square foot.

Coming nearer the ground, one seldom finds it reported that a tram-car has been blown over. Thus if P is the pressure in pounds per square foot on the car, then (Fig. 152) P $\times$ area $\times$ lever arm to the centre of pressure $=$ stabilising moment of W $\times$ half the gauge.

$$P \times 432 \times 8\tfrac{1}{3}\ \text{ft.} = 13^{T} \times 2{\cdot}375\ \text{ft., whence } P = 19\ \text{lb. per square foot.}$$

With the car empty the required pressure for overturning is only 15 lb. per square foot.

Most engineers can supplement this list from their own experiences to show that the wind must have been tempered to the earlier designers. Of several in the writer's experience the following is the most striking. On extending some existing workshops, which stood about 35 ft. high on a very exposed site at the estuary of a large river, the request was made that the extension should be a repeat of the old buildings which had stood for over half a century. The existing roof column shafts had the joist webs parallel to the gutter, and both columns and roof purlins were calculated to be stressed beyond the ultimate with a wind pressure considerably less than 30 lb. per square foot. The roof members were not quite so bad as they were stressed just over the elastic limit with a 30-lb. wind, but their riveted connections were so poor that the trusses were in effect no stronger than the columns.

It may be assumed, therefore, that the maximum gust velocity of the wind in Great Britain is in the region of 100 m.p.h., giving a maximum pressure of 31 lb. per square foot.

Maximal Velocities and Pressures in Other Countries.—In the north of the United States, Canada and Australia the horizontal wind pressure is taken at 30 lb. per square foot.

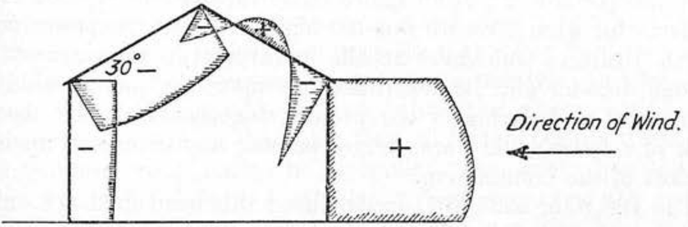
In the south of the United States and some parts of India fierce tornadoes or cyclones frequently occur. At the meeting of the American Association for the Advancement of Science in Philadelphia it was stated that a new record was set up by the hurricane at Miami on September 18th, 1926. At 7.40 a.m. the wind blew with a velocity of 132 m.p.h., and thirty-two minutes later the wind gauge was blown down. This velocity is equivalent to a pressure of 54 lb. per square foot. The amount of damage done by this storm was phenomenal.

Gust and Average Pressures.—The intensity of the wind pressure is seldom constant over a large area, for somewhere in the path of the wind will be found stream filaments or "pockets" of wind travelling at a higher velocity than the surrounding wind, and, therefore, exerting a higher pressure upon the gauge. On an average it would appear that the mean velocity is about 75 per cent. of the maximum gust velocity. The duration of the gust pressure is relatively small, somewhere about one or two seconds.

The large gauge of 300 sq. ft. at the Forth Bridge showed lower pressures than the small gauge of $1\frac{1}{2}$ sq. ft. which was inset in the larger gauge.

Suction or Negative Wind Pressures.—Among Dr. Stanton's earlier experiments were some dealing with the pressures on small models of buildings (*Min. Proc. Inst. C. E.*, Vol. CLVI.). Fig. 153 roughly indicates the distribution of the wind pressure on a model of a building when placed in the path of a wind travelling in the tunnel at a uniform velocity of about 17 m.p.h. The positive pressures are plotted on the outside of the building and the negative pressures inside.

A rather interesting example of suction was one where two particular panes of $\frac{1}{4}$-in. rough cast roof glass were always found



Normal Wind Pressures on a Model of a Building.
FIG. 153.

broken after a storm. The glazier, at last completely bewildered by the mysterious re-occurrences, reported the matter, and it was found that the glass and putty fillets were sucked clean out from the steel tee astragal after shearing the hardwood pins. On the latter being replaced by steel pins no further breakage was reported.

Despite the overwhelming evidence of suction on the leeward sides of roofs and buildings, etc., these are still universally designed to withstand only the full positive pressure on the windward side.

The Load Caused by Wind.—The possibility of the synchronisation of successive wind gusts with the period of vibration of a structure is so remote as to be entirely discountenanced.

Again, the maximum specified wind pressure, of 30 lb. say, may only occur once in the lifetime of the structure, perhaps never. It is, therefore, not out of the way to allow a slightly higher stress on this single occasion, but in no case should the elastic limit be approached too closely. The condition is on a par with the higher stresses allowed during erection, *e.g.*, a reinforced concrete pile usually acts as a column after driving, but when being slung into position by the crane it sometimes has to act as a beam carrying its own weight. Since this takes place only once, the usual working stresses of 600 and 16,000 lb. per square inch, for concrete and steel respectively, may be exceeded, in fact, sometimes the limit is set at 900 lb. per square inch for the concrete and 24,000 lb. per square inch for the steel.

From the foregoing it will be appreciated why practice never adds an impact allowance to the calculated wind stresses.

It has been advocated by some that wind should be taken as a live load of 40 lb. per square foot and then doubled to allow for impact. This is practically equivalent to taking a wind at 160 m.p.h., *i.e.*, about 30 m.p.h. more than the Miami record, a velocity which would permit of little life to exist on the face of the globe.

Conclusions.—Both Sir B. Baker and Sir J. Fowler recognised that the (Board of Trade) figure of 56 lb. per square foot of flat surfaces for wind pressure was too high for design purposes for the Forth Bridge. One must recall, however, the then recent Tay Bridge disaster and its reverberations upon the public mind, the immensity and daring of the project, together with the absolute lack of reliable wind data, before passing any comment upon the Report of the Commission.

The B.S.S. of 1923 (3/7) has reduced this wind load not only in direct numerical value, but also indirectly through the 25 per cent. increase allowance of Article 13, Part 3. Further, power is left to the engineer to reduce the specified figures if the position of the bridge warrants it.

The famous Quebec bridge was designed for a 30-lb. wind. This, in itself, means a large saving in weight, as can be seen from the fact that of the total estimated stress in one of the cantilever bottom booms of the Forth Bridge the specified wind contributed 47 per cent.

It has been shown that the larger the area the smaller is the average wind pressure upon it, which is one reason for the dual value for the wind pressure of the B.S.S. Also, storms in this country generally grow gradually in intensity, and so an engine driver receives ample warning before venturing over an exposed bridge.

The following is common practice in the design of buildings on ordinary town sites. Should the proposed site be one which is exceptionally exposed, the allowance for wind pressures can be increased accordingly.

Up to 50 ft. in height.—A horizontal wind pressure on the sides and ends of 15 lb. to 20 lb. per square foot, and a horizontal wind pressure of 25 lb. to 30 lb. per square foot for the roof.

50 ft. to 100 ft. in height.—20 lb. to 25 lb. (usual) up to 30 lb. for

sides and ends, and 30 lb. on the roof. No allowance is here made for negative pressures.

When a roof is to be erected on a very exposed site and is to be subjected to violent hurricanes the horizontal wind pressure may be increased to 40 lb. per square foot.

FACTORS OF SAFETY

The ultimate stress on a specimen under axial tension is the (breaking load) ÷ (the original net cross-sectional area).

The elastic limit is that stress which if exceeded will create a permanent set in the specimen. Below this point the specimen follows Hooke's law, which states that strain, or elongation, is proportional to the stress causing it.

To ensure that a structure will return to its original form on the withdrawal of the load the working stress must never exceed the elastic limit. If it were at all possible that every load and force could have its maximum resulting stress correctly evaluated, then the working stress could be just below the elastic limit. The proverbial straw more than this load, although far from breaking the structure, would hardly be advisable. The exact effect of every load cannot be definitely calculated, nor can even the correct numerical values of many loads be foretold, *e.g.*, live load stresses caused by locomotives, and wind loads. Again, we have stresses arising from the interaction of one part of a structure with another which can be estimated only approximately. Assumptions are also made in the calculations of primary stresses which are known to be not quite correct, *e.g.*, a joint with many wide-pitched rivets in it is assumed to be connected and held in place by one single and central pin. Further, there is the possibility of corrosion, imperfect workmanship, the fatigue of the material, etc., but sufficient reasons have been advanced to show that the working stress must be kept well below the elastic limit to allow for all contingencies.

Next comes the debatable point as to whether the working stress should be based upon the elastic limit or upon the ultimate strength. Now taking into account the wide variation which sometimes occurs between the calculated maximum stress and the actual maximum, as indicated by special tests, it is really immaterial whether we select as our datum the ultimate or the elastic limit, since these two bear a more definite and fixed relationship to each other than that which exists between the actual and the calculated stresses.

However, the factor of safety (f. of s.) is generally taken as (ultimate stress) ÷ (working stress) and for dead load is usually in the neighbourhood of 4. The ultimate tensile strength of mild steel for structural purposes (Appendix I. of the B.S.S., Parts 1 and 2) lies between the limits of 28 and 33 tons per square inch. Throughout the text, unless otherwise specifically stated, the average ultimate tensile stress is taken at 30 tons per net square inch, thus giving a working stress for dead load of $30 \div$ f. of s. of $4 = 7{\cdot}5$ tons per net square inch.

Unfortunately the working stresses for live loads cannot be settled so easily. An electric train has a different effect upon a bridge from one where the power is supplied by a steam locomotive, in fact, the various types of ordinary steam locomotives have vastly different effects at different speeds upon the bridge stresses. The type of bridge floor used has an important bearing upon the stresses in the main girders, *e.g.*, the ballasted track as against the tied floor. Next, there is an uncertainty as to the exact numerical value of all the effects now loosely but collectively grouped under the term "impact," and so it goes on.

The universal method with regard to bridge calculations (B.S.S. (3/5)), is to calculate the stresses due to the live loads as if they were momentarily static loads, and then to increase these stresses by an allowance for "impact." In the case of a small-span bridge or a cross girder the load comes upon it practically instantaneously. On the other hand, a large span only receives its maximal main boom live load stresses gradually as the train advances on to the bridge, and hence the impact factor is reduced as the span increases.*

Another fairly common method, and one which is perfectly satisfactory, is that used in shop design, which specifies that the :—

Wind stresses should have a factor of safety of 3.
Dead load stresses should have a factor of safety of 4.
Live load stresses should have a factor of safety of 5.

Thus if a member carries a wind load of 5^T, a dead load of 30^T, and a live load from the crane girders of 15^T, the procedure would be as follows :—

—	Actual Load, Tons.	F. of S.	Total Ultimate Load, Tons.
Wind . . .	5	3	15
Dead . . .	30	4	120
Live . . .	15	5	75
	50		210

* Many of the current ideas as to the effects of impact upon bridges will have to be revised in view of "*The Report of The Bridge Stress Committee*," issued by The Department of Industrial Research. This publication should be referred to by everyone interested in the design of railway bridges.

Hence the factor of safety to use is $210 \div 50 = 4{\cdot}2$. If the member is under tension then $f_t =$ ult. $\div$ f. of s. $= 30 \div 4{\cdot}2 = 7{\cdot}14^{\text{T}}$/sq. in., and the net area of metal required is $50 \div 7{\cdot}14 = 7$ sq. in. If a compression member, then the working stress is the ultimate as given by the strut formulæ divided by the f. of s. of 4·2.

Another method is to increase the actual live load in order to obtain an equivalent static load. A good example of this is the Ministry of Transport's Standard Load for Highway Bridges (June, 1922), where the actual wheel loads have been increased by 50 per cent., irrespective of speed, span of bridge, type of bridge floor, etc.

There are many other methods and formulæ used to arrive at the working stress, such as Launhardt-Weyrauch, the "Dynamic," etc., but in order not to confuse the issue the reader is referred to textbooks on the theory of structures.

REFERENCES

Wind Pressure

Stanton, T. E. *Resistance of Plane Surfaces in a Uniform Current of Air*. Min. Proc. Inst. C. E., CLVI.

Stanton, T. E. *Experiments on Wind-Pressure*. Min. Proc. Inst. C. E., CLXXI.

Stanton, T. E. *Report on the Measurement of the Pressure of the Wind on Structures*. Min. Proc. Inst. C. E., 219.

Remfry, D. H. *Wind-Pressures, and the Stresses caused by the Wind on Bridges*. Min. Proc. Inst. C. E., CCXVI.

Remfry, D. H. *Wind-Pressures at the Forth Bridge*. *Engineering of* February 28th, 1890.

Hunter, A. *Wind-Pressures at the Forth Bridge*. Transactions of the Junior Institute of Engineers, 1906.

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REFERENCES

Wind Pressure

Stanton, T. E. *Resistance of Plane Surfaces in a Uniform Current of Air.* Min. Proc. Inst. C. E., CLVI.

Stanton, T. E. *Experiments on Wind-Pressure.* Min. Proc. Inst. C. E., CLXXI.

Stanton, T. E. *Report on the Measurement of the Pressure of the Wind on Structures.* Min. Proc. Inst. C. E., 219.

Remfry, D. H. *Wind-Pressures, and the Stresses caused by the Wind on Bridges.* Min. Proc. Inst. C. E., CCXVI.

Remfry, D. H. *Wind-Pressures at the Forth Bridge. Engineering* of February 28th, 1890.

Hunter, A. *Wind-Pressures at the Forth Bridge.* Transactions of the Junior Institute of Engineers, 1906.

CHAPTER X

BEAMS

$$\frac{M}{I}=\frac{f}{y}=\frac{E}{R} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad 1$$

Where $M =$ Bending moment in inch tons (or foot tons),

$I =$ Moment of inertia of beam's cross-section in inches^4 (or feet^4).

$f =$ Stress in tons per square inch (or tons per square foot).

$y =$ Distance from neutral axis to the fibre considered, in inches (or feet).

$E =$ Young's modulus in tons per square inch (or tons per square foot).

$R =$ Radius of curvature of the bent beam, in inches (or in feet).

Notwithstanding that this formula is perhaps the most important one employed in the design of structures, beginners often make absurd mistakes through neglecting to specify the units employed. This is not altogether their fault, because many text-books on mechanics prove the formula as a mathematical problem without clearly stating the units which are employed, although units are of paramount importance to the engineer. In steelwork calculations in this country stresses are always given in tons per square inch, and for reinforced concrete and timber work in pounds per square inch. If f is required in tons per square inch, then all the other items of equation **1** must be expressed in tons and inch units.

Thus $$\frac{M}{I} \text{ is } \frac{\text{M in inch tons}}{\text{I in inches}^4}=\frac{M}{I}\left(\frac{\text{tons}}{\text{inches}^3}\right).$$

Similarly, $$\frac{f}{y}=\frac{f \text{ in tons/sq. in.}}{y \text{ in inches}}=\frac{f}{y}\left(\frac{\text{tons}}{\text{inches}^3}\right),$$

since " per " or " / " is equivalent to the sign of division. The words ton and inch can be " cancelled out top and bottom " as with ordinary arithmetical figures. Again,

$$M=\frac{fI}{y} \text{ is in } \frac{(\text{tons/sq. in.}) \times \text{inches}^4}{\text{inches}} = \text{tons . inches} \quad . \quad . \quad . \quad . \quad 2$$

When y is the distance to the extreme fibre from the neutral axis, the term $\frac{I}{y}$ is constant for that particular section. It can therefore be represented by one symbol Z, known as the section modulus.

$$\therefore \qquad \frac{M}{f} = \frac{I}{y} = Z \text{ in inches}^3 \qquad \ldots\ldots \quad \mathbf{3}$$

In the cases of joists and channels the distance **y** from the XX neutral axis to the extreme fibres is the same, and, therefore, the fibre stress is numerically the same (but opposite in sign, *i.e.* tension

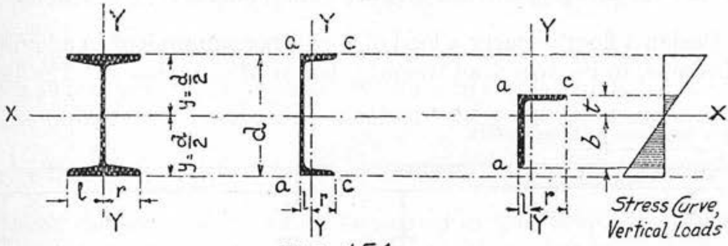


FIG. 154.

and compression) on the top and bottom flanges, by equations 1 and **3**.

The angle, however, has two values for **y**, viz. **t** and **b**. Hence, Z top or $Z_t = \frac{I}{y_t}$ and Z bottom $= Z_b = \frac{I}{y_b}$ from equation **3**.

The fibre stress on the top fibre of the angle is $f =$ bending moment $\div$ section modulus $= M \div Z_t$, and on the undermost fibre it is $M \div Z_b$.

If the load, instead of acting vertically along the yy axis, now acts horizontally along the XX axis, the fibre stress is numerically the same at all four corners of the joist. With the channel and the angle there is a Z_l and a Z_r, consequently the stress intensity at the points **a** is the same, but is of different value and of opposite sign to the stress at points **c**. In the listed moduli of unsymmetrical sections it is always the smaller Z which is given.

Approximate Section Modulus of a Joist.—In the case of the new British Standard Joists it will be found that if the depth in inches is multiplied by the weight per foot in pounds, then the number so obtained is about ten times the (XX) modulus of the section ; there is no question of units involved here, as the rule is entirely empirical.

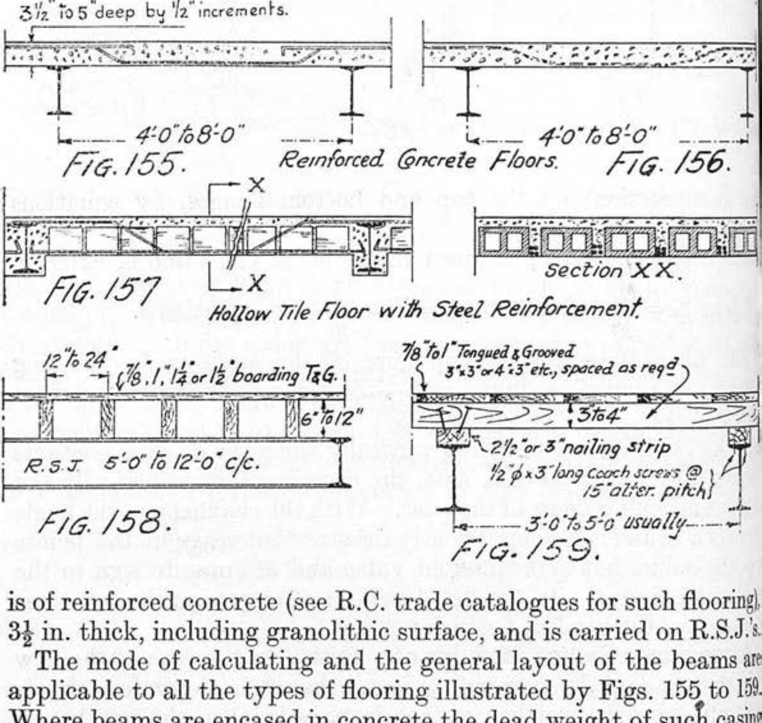
The Z of a $14'' \times 5\frac{1}{2}'' \times 40$ lb. R.S.J. is 53·87, against $14 \times 40 \div 10 = 56$ by the rule. The multiplier 10 is the average (9·96) and really varies from 10·6 to 9·5, with an error not larger than 6 per cent. The rule is a useful one for mental approximations.

Approximate Section Modulus of a Channel.—With channels the constant is 11 instead of 10. (It varies from 10·2 to 12·3, average for all channels is 11·1.)

$12'' \times 4'' \times 31{\cdot}33$ lb. channel has the listed modulus of 33·5, whereas the rule gives it as $12 \times 31{\cdot}33 \div 11 = 34{\cdot}1$. This rule, however, is not nearly so useful as the joist one, which only involves the "shifting of the decimal point one place."

EXAMPLE OF SIMPLE BEAM CALCULATIONS: FLOORS

Design a floor to carry a load of 1 cwt. per square foot in addition, of course, to its own dead weight; factor of safety = 4. The floor



is of reinforced concrete (see R.C. trade catalogues for such flooring), $3\frac{1}{2}$ in. thick, including granolithic surface, and is carried on R.S.J.'s.

The mode of calculating and the general layout of the beams are applicable to all the types of flooring illustrated by Figs. 155 to 159. Where beams are encased in concrete the dead weight of such casing must be taken into account. Suspended or false ceilings should have their weight estimated at the rate of 8 to 12 lb. per square foot of ceiling area. (Reinforced concrete 150 lb., and ordinary concrete 140 lb. per cubic foot.)

In laying out the key plan of the floor beams previous to calculating it is economical to have :—

(1) The major portion of the steelwork of small span for easy handling.

(2) Equal spacing of beams.

(3) If possible, no concentrated load at mid-span.

Working Stresses.—The permissible tensile stress = the average ultimate ÷ 4. The ultimate lies between 28 and 33 tons per square inch (B.S.S., Parts 1 and 2), thus giving an average ultimate of 30·5 tons per square inch, and, hence, the permissible tensile stress may be taken at 30·5 ÷ 4 = 7·6, or, as is usually adopted, 7·5 tons per square inch.

The concrete slab floor being cast *in situ* upon the steel beams exerts an adhesive and frictional resistance to any lateral movement of the upper or compressive flanges. In the formula $f_c = f_t\left(1-0{\cdot}01\,\frac{l}{b}\right)$ of the B.S.S. (3/18) the term l is zero, and hence $f_c = f_t$.

Adopting the same ratios as are contained in the B.S.S., the other permissible stresses are :—

$f_t = f_c$	=	tons per sq. in.	7·5
f_w = shear on web = $\frac{5}{8}$ of f_t	=	,,	4·69
f_s = single shear on rivets = $\frac{3}{4}$ of f_t	=	,,	5·63
f_b = bearing value of rivets = $1\frac{1}{2}$ of f_t	=	,,	11·25

The beams are free-ended or simply supported, the end cleats being too shallow to afford any fixity. The maximum bending moment, therefore, occurs at mid-span and the maximum shear at the ends, both occurring simultaneously when the floor is completely loaded.

Light Beams or Stringers.—Span 12 ft. 6 in. at 5 ft. c/c., Fig 160.

Loading.

Superimposed load	= 112 lb. per sq. ft.		
$3\frac{1}{2}''$ R.C. floor = $(3\frac{1}{2} \div 12) \times 150$	= 44 lb. per sq. ft.	= lb./sq.ft.	156
External load on one stringer	= 156 lb./sq. ft. × 5′ × 12·5′	= tons	4·35
Maximum B.M. from above	= $Wl \div 8$ = 4·35ᵀ × 12·5′ × 12″ ÷ 8	= in. tons	81·6

[81·6 ÷ f = approximate modulus, and so an idea of the weight of the beam is obtained from the section list.]

Weight of beam	= 18 lb./ft. × 12·5′	= tons	0·1
Maximum B.M. due to beam's own weight	= 0·1ᵀ × (12·5′ × 12″) ÷ 8	= in. tons	1·9

Total B.M. $= 81{\cdot}6 + 1{\cdot}9$ $=$ in. tons 83·5

Total Max. Shear $=$ reaction $= \frac{1}{2}\,(4{\cdot}35^{\text{T}} + 0{\cdot}1^{\text{T}}) =$ tons 2·25

Modulus req. $= \text{M} \div f_t = 83{\cdot}5 \div 7{\cdot}5$ $=$ in.^{3} 11·1

Web area req. $=$ shear $\div f_w = 2{\cdot}25 \div 4{\cdot}69$ $=$ sq. in. 0·48

Section

Try 1 R.S.J., $7'' \times 3\frac{1}{2}'' \times 15$ lb. (too light) Z $=$ in.^{3} 10·3

Adopt 1 R.S.J., $8'' \times 4'' \times 18$ lb. Z $=$,, 13·9

Web area, allowing for 2″ notch at top
$= (8'' - 2'') \times 0{\cdot}28''$ $=$ sq. in. 1·68

Deflection

$$= \frac{5\,\text{W}l^3}{384\,\text{EI}} = \left(\frac{5 \times 4{\cdot}5 \times 150^3}{384 \times 13{,}000 \times 55{\cdot}6}\right)\left(\frac{\text{tons} \times \text{ins.}^3}{\text{tons per sq. in.} \times \text{ins.}^4}\right)$$

$=$ in. 0·27

Where W $=$ total load in tons ; E $= 13{,}000$ tons per square inch; I $= 55{\cdot}6$ in.^{4} obtained from the list of properties of sections.

For plaster work the deflection $\varDelta$, is $\ngtr$ span $\div$ 480 $=$ 150 $\div$ 480 $= 0{\cdot}31''$, although span $\div$ 360 is quite a common figure.

The beam can be notched safely at the end because the top flange has no B.M. to carry at this point (Figs. 163 and 164).

Main Beams between Columns.—Span 25 ft. @ 12 ft. 6 in. c/c. (Figs. 160 and 161.)

The loading on this beam consists of four pairs of concentrated loads (eight stringer reactions), in addition to a distributed load.

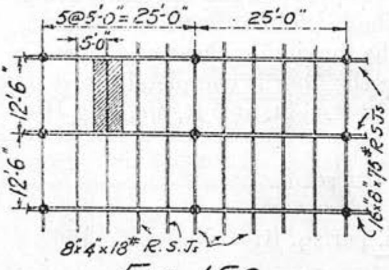


FIG. 160.

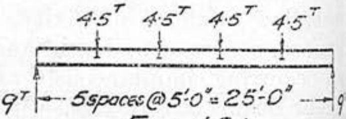


FIG. 161

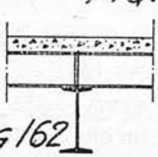


FIG 162

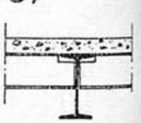


FIG. 163.

(1) from the beam itself, (2) from the concrete floor. The latter is small and its exact value cannot be definitely stated. Further, it can only be taken into account if the concentrated loads from the stringers be reduced proportionately. Since a concentrated load gives a larger B.M. than the same load uniformly distributed, it is safer, and at the same time easier, to neglect any local distributed load acting directly from the floor concrete on the main beam. The concentrated load from the floor stringers will be taken as if the

stringer carried all the load on the shaded area of Fig. 160. This is equivalent to assuming that the stringers pass over the main beam (Fig. 162) instead of bosoming into it (Fig. 163). Further, the possibility of all the floor space being covered with goods or people weighing 1 cwt./sq. ft. is remote, as access passageways must be maintained; but the position of these spaces will vary from time to time, so that it is but reasonable to assume all the floor loaded and thus considerably simplify the design.

Fig. 162 suggests that if the stringers were carried continuously over the main beam a lighter stringer would result, from the fact that the maximum B.M. in a beam continuous over several spans is less than that in the simply supported single span. This undoubtedly is true, but such an advantage can only be gained at the expense of head room, which is a more vital factor in the design.

The span of the beam is taken as being centre to centre of column shafts. This is on the safe side and permits the use of any type of column, wide or narrow; see also the B.S.S. (4/2).

Loading :—

The end reactions from each pair of stringers $= 2 \text{ @ } 2{\cdot}25^{\text{T}}$	$=$ tons	4·5
Maximum B.M. due to above panel loading (Fig. 161) is $(9^{\text{T}} \times 10' - 4{\cdot}5^{\text{T}} \times 5')$ $= 67{\cdot}5$ ft. tons	$=$ in. tons	810
(Approximately $Z = 810 \div 7{\cdot}5 = 108$; roughly the R.S.J. weighs 65 lbs.)		
Weight of beam self $= 65 \times 25 = 1625$ lb. (neglecting cleats)	$=$ tons	0·72
Maximum B.M. due to beam self $= Wl \div 8 =$ $0{\cdot}72 \times 25 \times 12 \div 8$	$=$ in. tons	27
Total Max. B.M. at mid-span $= 810 + 27$	$=$,,	837
Modulus required $= 837 \div 7{\cdot}5$	$=$ in.^{3}	112
Web area required $=$ end shear $\div f_w$ $= (9^{\text{T}} + \frac{1}{2}$ of $0{\cdot}72^{\text{T}}) \div 4{\cdot}69^{\text{T}}$/sq. in.	$=$ sq. in.	2
Modulus given. One R.S.J., $20'' \times 6\frac{1}{2}'' \times$ 65 lb.	$Z =$ in.^{3}	122·6
Note.—There is practically no diminution in the value of the listed section modulus as the beams are holed in the web and not in the flanges.		
Web area given, $20'' \times 0{\cdot}45''$ (B.S.S. 4/4))	$=$ sq. in.	9

Deflection.—Since the joist depth is $\frac{1}{15}$ of the span, and the maximum fibre stress is slightly less than 7·5 tons/sq. in., the beam need not be investigated for deflection. See a later article in this chapter under the heading of Deflection of Beams, Ratio of Bea Depth to Span. A quick slide rule check, however, is obtaine by assuming all the load uniformly distributed and then using th standard form of $\frac{5}{384} \frac{Wl^3}{EI} = \frac{5 \times 18{\cdot}72 \times 300^3}{384 \times 13{,}000 \times 1226} = 0{\cdot}413''$.

The deflection obtained by this approximation is on the low sid but it is well under the permissible of span $\div$ 480 = 0·625″, and indicates that the beam is sufficiently rigid. The presumably tru deflection is 0·496 in. as found by the more intricate calculation Fig. 168. The actual deflection may be slightly less than the la figure owing to the monolithic nature of the floor.

Alternative Section is the R.S.J.

$16'' \times 8'' \times 75$ lbs., whose Z is = in.^{3} 121·7,
and web area is $= 16 \times 0{\cdot}48$ = sq. in. 7·68.

This joist is preferable to that found above, although it weig 10 lb. more per foot run. It is 4 in. less in depth than the 20-i R.S.J., which means a 4-in. saving on the height of the buildin As the 10 lb. extra per foot run hardly affects the B.M. or end she already found, the only point to check is the deflection, which, sin the depth is now less, will be greater than that for the previou joist; *i.e.*

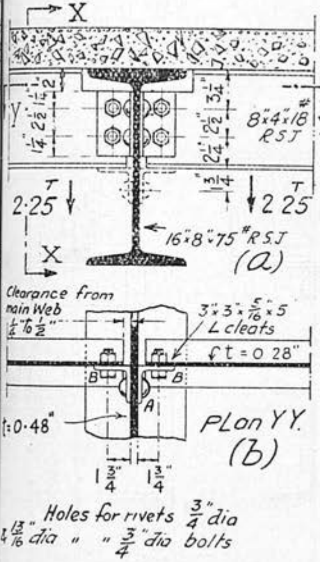
$= 0{\cdot}413 \times$ inverse ratio of the inertias $= 0{\cdot}413 \times 1226 \div 973{\cdot}9 = 0{\cdot}52''$

Similarly, the true deflection $= 0{\cdot}496 \times 1226 \div 973{\cdot}9 = 0{\cdot}624''$
which is just under the permissible deflection of 0·625″

Adopt $16'' \times 8'' \times 75$ lb. R.S.J. for the main beams.

Fastenings.—Stringers to main beam. A rough guide to cle thickness is that these should never be thinner than the web of th thinner joist to which they are riveted. They act as diminuti cantilevers and can be so calculated, but an experienced design seldom, if ever, does so. Where there is a cleat on each side of th joist web each may be of the same thickness as the web; if onl one cleat, then it should be made slightly thicker than the joist we and if the cleat has two rivet lines per leg, the cleat thickness shou be still greater because of the longer lever arm to the centre of gravi of the rivet group on the outstanding leg.

Fig. 164.—The 2-in. deep notch in the top flange of the light joi permits it to run into the web of the heavy joist without fouling (s Table 13, Vol. III., for clear depth of joist and channel webs). Similarl anything riveted to the web of the light joist must stop short at point 1 in. above the bottom flange so as to clear the bottom ro fillet. The clear depth of the cleats is thus limited to 5 in., and th in turn, limits the rivet diameter to $\frac{3}{4}$ in. or $\frac{13}{16}$ in. Rivets $\frac{7}{8}$ in. in diameter would require a total clear depth of $1\frac{1}{2}$ diameters + 3 diameters + $1\frac{1}{2}$ diameters $= 6 \times \frac{7}{8}'' = 5\frac{1}{4}''$ by the B.S.S. (4/25 and 4/27). The cleats will be riveted to the main beam previous to dispatch, and open holes will be left in the web of the light joist for site rivets or bolts; the latter fastening is the more probable.



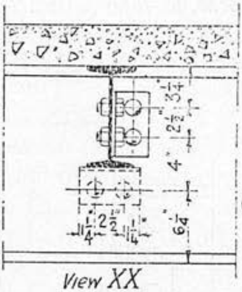


FIG 164.

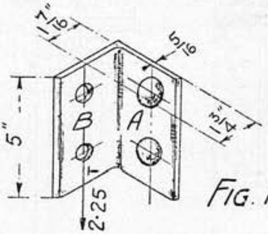


FIG. 165.

This allows the main beams to be erected, and then afterwards the light beams are swung into position.

$\frac{3}{4}$ in. diameter rivet values :—S.S. $= 2{\cdot}49^{T}$, D.S. $= 4{\cdot}97^{T}$, bearing, $0{\cdot}28'' = 2{\cdot}36^{T}$ and $0{\cdot}48'' = 4{\cdot}05^{T}$; see Table 12, Vol. III.

Using a single cleat at B places the rivets thereat in S.S. or 0·28 in. bearing; the joist web is thinner than the cleat.

Number of rivets required $= 2{\cdot}25^{T}$ load $\div\ 2{\cdot}36^{T} = 1$.

If black bolts are used instead of rivets increase this number by 20 per cent. (B.S.S. (3/18)), *i.e.*, say 2. In any case never use less than two fastenings.

As both bolts and rivets are taken at $\frac{3}{4}$ in. finished diameter, the holes for these must be $\frac{13}{16}$ in. and $\frac{3}{4}$ in. diameter respectively. Shop work would be facilitated if both sets of holes were made $\frac{13}{16}$ in. diameter, *i.e.*, $\frac{3}{4}$ in. diameter bolts and $\frac{13}{16}$ in. diameter rivets.

The cleat adopted will be $3'' \times 3'' \times \frac{5}{16}'' \times 5''$, which has the rivet lines in each leg sufficiently far apart to prevent the fouling of the rivet heads at A with the bolts at B, as the depth of 5 in. does not

permit the reeling of the A fastenings with the B fastenings. T
load on the A fastenings is twice $2{\cdot}25^{T} = 4{\cdot}5^{T}$, and these are
D.S. or 0·48-in. bearing. Giving two rivets at A provides for
direct load of $2 \times 4{\cdot}05^{T} = 9{\cdot}1^{T}$, which is much in excess of t
actual direct load of $4{\cdot}5^{T}$.

Fig. 165.—Calculating the cleats as cantilevers rigidly held to t
main joist and carrying a concentrated load at the bolt lines B
$2{\cdot}25^{T}$, shows a B.M. at face A of $2{\cdot}25^{T} \times 1\frac{7}{16}'' = 3{\cdot}23$ in. tons. T
resisting modulus at face A is that of a flat 5 in. deep $\times \frac{5}{16}$ in. thick
$\frac{1}{6} \times \frac{5}{16} \times 5^2 =$ in.^{3}, 1·30. The maximum fibre stress at the t

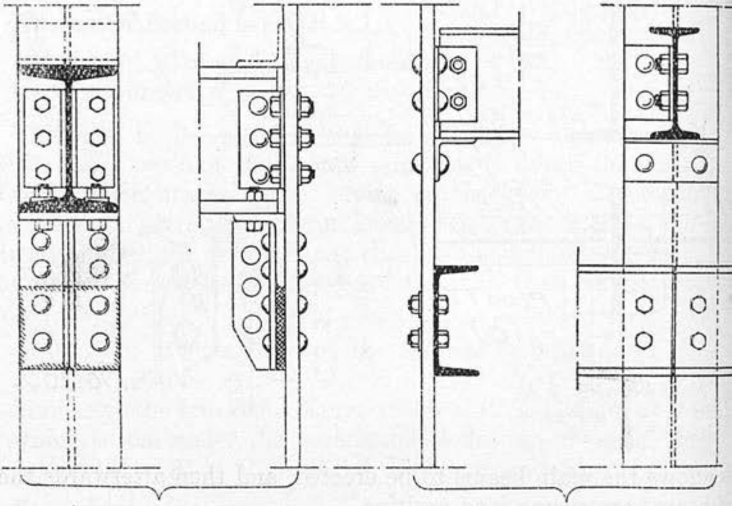


FIG. 166. FIG. 167.

and bottom of the cleat where the outstanding leg meets face A
$M \div Z = 3{\cdot}23 \div 1{\cdot}3 = \pm\ 2{\cdot}5$ tons/sq. in.

Landing, or shelf, cleats are useful in supporting the light jo
during erection. Sometimes their use becomes imperative whe
sufficient number of bolts to carry the end shear cannot be pla
in the vertical cleats, and the end shear has to be countered by b
the shelf and vertical cleat fastenings ; in the case under discuss
such a necessity does not arise. A $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ or $3'' \times 3'' \times$
angle, 5 in. long, with two rivets (indicated in broken line in Fig. 1
will easily support the dead load of the stringer until bolting u
completed. The bolting up of the bottom flanges of the light jo
to these shelf cleats is a matter of opinion, but if carried out bra
the whole structure laterally during erection. Once the conc

floor is laid the necessity for such bolts hardly exists. One or two bolts may be used in conjunction with bevelled washers on the sloping face of the bottom flange of the light joist.

If four vertical cleats instead of two had been used at the joint (*i.e.*, one cleat on each face of the thin webs) no gain in rivet values at B would have been obtained, since the light joist web is so thin that the bearing value is actually less than the single shear value.

Main Beams to Columns.—The number of bolts required to transfer the 9 tons end reaction of the main beam into the column shaft is found in a manner similar to that used for the stringers to main beams. A few typical beam to column details are illustrated in Figs. 166 and 167.

Flange Width.—The upper or compression flanges of all beams and joists tend to buckle sideways because of the strut action to which they are subjected. The longer and narrower these flanges are the more prone are they to crumple, and to counteract this the working compressive stress is lowered as the flange width decreases. In the present instance all the upper flanges are well supported laterally by the frictional and adhesive contact with the concrete floor. Taking the working adhesive stress at 100 lb. per square inch this supporting force alone, per foot run of the 8″ × 4″ × 18 lb. R.S.J., is = 4″ × 12″ × 100 lb./sq. in. = slightly over 2 tons. If the upper flanges had no lateral support the f_c would be reduced to $f_t\left(1 - 0{\cdot}01\dfrac{l}{b}\right)$ (B.S.S. (3/18)). Suppose, for example, that the main joist had been as in Fig. 162 with the flange supported every 5 ft. by the stringers (if bolted to main beam), then $f_c = 7{\cdot}5\left(1 - 0{\cdot}01\dfrac{60}{8}\right) =$ 6·94 tons/sq. in. The necessary modulus is now settled from the point of view of compression and is B.M. ÷ 6·94 = 837 ÷ 6·94 = 120·6 in.^{3} The 16″ × 8″ × 75 lb. R.S.J. is still satisfactory, as its modulus is 121·7 in.^{3}

DEFLECTION OF BEAMS

When the loading of a beam or girder is irregular, and of a form not covered by the standard formulæ, probably the easiest way to evaluate the deflection is as follows.

Load the beam or girder with the actual B.M. curve and find the resulting or second B.M. diagram due to this loading. This secondary B.M. curve is the deflection diagram for the beam, and an ordinate at any point represents, to scale, the vertical deflection of the girder at that point.

Thus in Fig. 168 the load diagram of (*a*) shows the joist as having its own weight concentrated at the panel points instead of being uniformly distributed. This causes but a very slight difference in the value of the maximum B.M., as witness 836 against 837 previously used. The area of portion $A_1 = \frac{1}{2} \times 60'' \times 557$ in. tons $= 16{,}710$ in.^{2} tons, and the centre of gravity of the area is 110 in. from the centre line; similarly with the two remaining areas. The reactions R_L and R_R are now each equal to $A_1 + A_2 + A_3 =$ 83,580 in.^{2} tons. From the symmetry of the figure the maximum

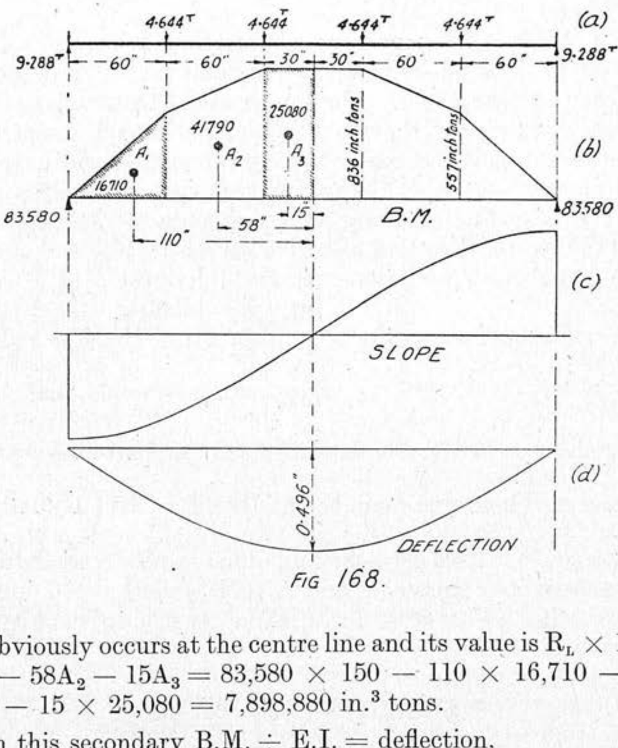


FIG 168.

B.M. obviously occurs at the centre line and its value is $R_L \times 150'' - 110A_1 - 58A_2 - 15A_3 = 83{,}580 \times 150 - 110 \times 16{,}710 - 58 \times 41{,}790 - 15 \times 25{,}080 = 7{,}898{,}880$ in.^{3} tons.

Then this secondary B.M. $\div$ E.I. = deflection,

$$\Delta = \frac{7{,}898{,}880 \times \text{in.}^3 \text{ tons.}}{13{,}000 \text{ tons per square inch} \times 1{,}226 \text{ in.}^4} = 0{\cdot}496 \text{ in.}$$

Remember that "per" is equivalent to the sign $\div$ and that the units "cancel out" as with ordinary numbers.

In the present instance neither the slope curve, *i.e.*, the secondary curve of shear obtained from the primary B.M. as loading, nor the deflection curve require to be drawn since, from symmetry, the maximum B.M. occurs at the centre line. It will be recalled that when the primary shear curve cuts the base line, *i.e.*, zero value, the primary B.M. curve registers a maximum value. Similarly with the secondary shear and B.M. curves, for where the slope curve cuts its base line the deflection is a maximum, as will be seen from the figure. This property is of value when the loading is asymmetrical, because if the secondary shear or slope curve is sketched out the position of maximum deflection is indicated and the secondary B.M. or deflection calculated for this one point only. It is to be emphasised that the point of maximum deflection does not necessarily occur under a concentrated load. Thus if a single concentrated load is applied at a quarter point of the span of a beam the maximum deflection occurs neither at the centre line nor at the load point, but at a point distant $0{\cdot}06 \times$ span from the centre line and on the same side of the centre line as the load.

Ratio of Beam Depth to Span is determined, apart from æsthetics, by the permissible deflection. Usually the ratio is about $\frac{1}{15}$ for $\frac{\text{depth}}{\text{span}}$ for heavy beam work, $\frac{1}{20}$ for medium and $\frac{1}{25}$ for light work. The following rule is, however, more satisfactory.

Let W = total load in tons; l = span in inches; E = Young's modulus of $13{,}000^{\text{T}}$/sq. in. for beams; while $12{,}000^{\text{T}}$/sq. in. is usually adopted for built-up girders.

I = the moment of inertia in in.^{4}; f = stress in tons/sq. in.
Y = the distance in inches to fibre from the neutral axis. If to the outermost fibre then,
Y = $\frac{1}{2}$ beam depth = $\frac{1}{2}$ D.
M = bending moment in inch tons = $Wl \div 8$ and $Wl \div 4$ in the following examples:—

Since $$\frac{M}{I} = \frac{f}{y} \therefore M = \frac{If}{y} = \frac{2If}{D} \quad . \quad . \quad . \quad . \quad . \quad \mathbf{1}$$

For a uniformly distributed load

$$\Delta = \frac{5Wl^3}{384EI} = \frac{Wl}{8}\left(\frac{5l^2}{48EI}\right) \quad . \quad . \quad . \quad . \quad . \quad \mathbf{2}$$

$$\text{Now } \frac{Wl}{8} = M = \frac{2If}{D} \text{ by } \mathbf{1} \quad \therefore \Delta = \frac{2If}{D}\left(\frac{5l^2}{48EI}\right) = \frac{fl^2}{4{\cdot}8ED} \quad . \quad \mathbf{3}$$

If f be limited to $7{\cdot}5^{\text{T}}$/sq. in., and Δ to (span $\div$ 360), equation **3** can be written

$$\frac{l}{360} = \frac{fl^2}{4{\cdot}8ED} \text{ or } D = \frac{562{\cdot}5l}{E} \quad . \quad . \quad . \quad . \quad . \quad \mathbf{4}$$

For beams,

$$\text{the depth } D = 562{\cdot}5l \div 13{,}000 = \text{span} \div 23 \quad . \quad . \quad 5$$

For built-up girders,

$$\text{the depth } D = 562{\cdot}5l \div 12{,}000 = \text{span} \div 21 \quad . \quad . \quad 6$$

Similarly for a concentrated load at the centre of the span,

$$D = \frac{450l}{E} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad 7$$

For beams,

$$\text{the depth } D = \text{span} \div 29 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad 8$$

For built-up girders,

$$D = \text{span} \div 27 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad 9$$

Rule.—Therefore, it can be stated that if the extreme fibre stress does not exceed $7{\cdot}5^{T}$/sq. in., and if the beam or girder depth is not shallower than span $\div$ 20, then the deflection need not be calculated, since it will not exceed span $\div$ 360.

LINTEL BEAMS

The following example should present no difficulty if the design of the floor beams has been followed. Find a suitable rolled steel section to carry a masonry wall 12 in. thick by 15 ft. high over a 15 ft. clear opening (masonry at 150 lb. per cubic foot).

Permissible stresses.	$f_t = 7{\cdot}5$ tons per net sq. inch.		
	$f_c = f_t\left(1 - 0{\cdot}01\,\dfrac{l}{b}\right)$.		
	f_w on web $= \frac{5}{8} f_t = 4{\cdot}69$ tons per gross square inch.		
	Bearing pressure on sandstone end bearing blocks	= tons/sq. ft.	16
Effective span.	Allow a 1 ft. length of bearing at each end. Span c/c of bearings	= ft.	16
Weight of wall	$= 16' \times 15' \times 1' \times 150$ lb./ft.^{3} $\div$ 2240 lb.	= tons	16
Weight of beam	is estimated at	,,	0·5
Total weight	uniformly distributed	= ,,	16·5
Maximum B.M.	$= Wl \div 8 = 16{\cdot}5 \times (16 \times 12) \div 8$	= in. tons	396
Z required	for tension $= 396 \div 7{\cdot}5$	= in.^{3}	53
Shear area required	= end shear $\div f_w = (\frac{1}{2}$ of $16{\cdot}5^{T}) \div 4{\cdot}69$	= sq. in.	1·8
Try a $10'' \times 8'' \times 55$ lb. R.S.J.	Broad flange to give good seating, Z	= in.^{3}	57·74

5 Check for f_c $= 7{\cdot}5\left(1 - 0{\cdot}01\,\dfrac{16 \times 12}{8}\right)$ = tons/sq. in. 5·7

6 Z required for compression $= 396 \div 5{\cdot}7 =$ in.^{3} 69·5

This section is not suitable.

7 Now try a 12″ × 8″ × 65 lb. R.S.J. (Fig. 169), Z = in.^{3} 81·3

Since the flange is still 8 in. wide this beam is satisfactory from the point of view of compres-

8 sion, where the required Z = ,, 69·5

Shear area = beam depth × web thickness

9 given $= 12 \times 0{\cdot}43$ = sq. in. 5·16

Δ permissible $=$ span $\div 480 = 16 \times 12 \div 480 =$ in. 0·40

Δ actual $= \dfrac{5 \times W \times l^3}{384 \times E \times I}$

$= \dfrac{5 \times 16{\cdot}5 \times (16 \times 12)^3}{384 \times 13{,}000 \times 487{\cdot}8} =$,, 0·24

Bearing at ends. Given 1 ft. long × 8 ins. flange width on bed block = sq. ft. 0·66

Required, for sandstone, $= (\frac{1}{2}$ of $16{\cdot}5^{T}) \div 16^{T}$/sq. ft. = ,, 0·52

Alternative Section. Two R.S.J's. side by side, 12″ × 5″ × 30 lb.; Z = 2 @ 34·5 = in.^{3} 69

When tied as in Fig. 170 the width of top flange is $10\frac{1}{2}$ in. to be used in the formula for f_c.

The two upper flanges are tied to each other by means of the brickwork, which serves as a continuous tie plate fastened to the

FIG. 169

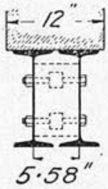


FIG 170

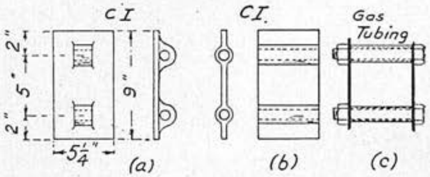
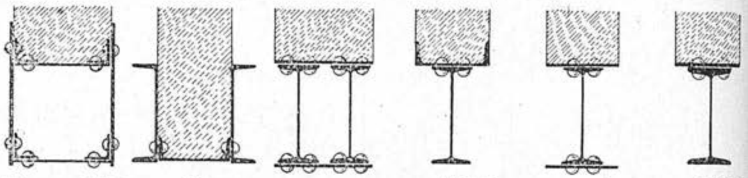


FIG 171

steel by the adhesion of the mortar. In addition, cast iron or mild steel (pressed) separators will be used at 4 ft. centres. The usual spacing runs from four to six times the beam depth. A separator will be placed at each bearing, so that there will be a total of five separators in the lintel beam. For sizes and weights of these separators see trade catalogues.

The separators keep the beams together laterally, but little faith can be placed upon their acting as diaphragms equalising the load between the joists. Each beam, due to the clearance of the bolts in the holes of the casting, is free to deflect without obtaining much help from its neighbour, and should one beam deflect more than the other it will result in a cracked wall. Because of this some designers economise in weight by using bolts placed inside gas tubing for



FIGS. 172. 173. 174 175 176 177

TYPES OF LINTEL GIRDERS.

separators; they are just about as effective and cost less than the castings, which are illustrated in Fig. 171.

Other types of built-up lintel girders are shown in Figs. 172 to 177. Generally speaking, the first three are for large spans. When the section is unsymmetrical the centre of gravity of the section must be found, and then the Z top and Z bottom. Fig. 175 is the cross-section of a simple and very efficient form of lintel girder, the Z's of which are easily obtainable. The pitch of the rivets in the built-up

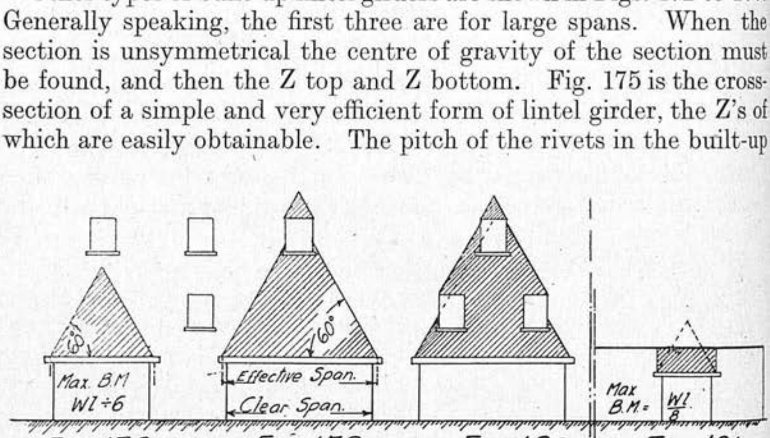


FIG. 178 FIG. 179 FIG. 180 FIG. 181

sections are found from the formula $q = VG \div Ib$, a numerical example of which is given further on in the present chapter.

When lintel girders carry a wall unbroken by windows there is an arch action within the brick or masonry work which relieves the girder of some of the load. The assumption usually made is that the girder carries only the hatched equilateral triangle of wall (Fig. 178). The remaining figures (179 to 181) are special cases of this rule ; the shaded areas illustrate the active loads on the lintel girders. When building over a steel lintel it is better not to raise the brickwork too rapidly, for with a green wall there is the possibility of more than the equilateral triangle of loading acting on the beam.

The maximum B.M. with triangular loading is $Wl \div 6$, where W is the total load. The foregoing example took a liberal view of the possible load and is therefore on the safe side ; the B.M. with triangular loading is 236 in. tons + 12 in. tons due to dead weight of lintel itself ($Wl \div 8$).

DECREASING AND INCREASING THE STRENGTH OF A BEAM

Beams with Holes.—The listed moduli and moments of inertia are for gross sections without holes. Fig. 182 shows a joist with holes in the upper or compression side of the neutral axis, then :—

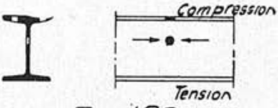


FIG. 182

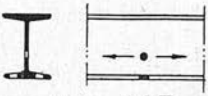


FIG. 183.

(1) If these are rivet-filled no deduction need be made from the gross Z and M. of I. of the beam, as the rivets, on the assumption that they completely fill the holes, transfer the thrust across from one side of the gap to the other. (2) If bolt filled (with a $\frac{1}{16}$ in. clearance in the diameters) the holes are not plugged, and deduction must be made from the Z and M. of I. The holes in the joist of Fig. 183 are on the lower or tension side of the N.A. In this case, as indicated by the arrows, the opposite sides of the holes tend to separate, and no matter whether the holes be rivet- or bolt-filled, deduction must be made from the gross Z and M. of I. of the beam.

In the cross-sections of both of these figures the N.A. and C. of G. line is no longer half-way up the section, but is nearer the unholed flange, where the heavier mass of metal lies ; while at a section past the holes the N.A. and C. of G. line returns to the mid-depth position. It is common practice to take the N.A. as always being at mid-depth, where it really lies for the major length of the beam, so eliminating the irksome calculations necessary in finding the new C. of G. and M. of I. of the beam.

When beams are holed on the lower side only, the foregoing practice is equivalent to deducting a similar set of holes from the upper portion of the beam, although none may exist there. When the holes are in the upper portion of the beam, and none out of the lower portion, then the gross Z and gross M. of I. may be used provided the holes are rivet-filled ; if bolt-filled a similar set of holes should be assumed out of the lower portion of the beam, see method (*c*) under.

To appreciate this statement better, consider the ideal beam of

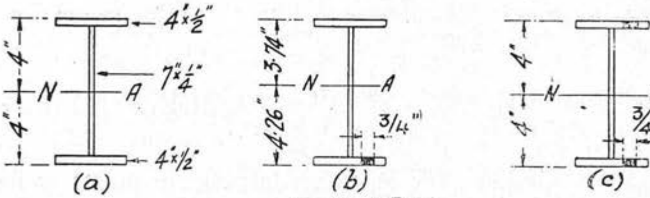


FIG. 184.

Fig. 184, which has been used in place of an R.S.J. because of the facility with which the M. of I. can be found.

(*a*) Gross Section.—M. of I. = 63·47 in^4. Z, top and bottom = in.^{3} 15·87

(*b*) Net Section.—M. of I. = 57·76 in^4. Z top = ,, 15·45
and Z bottom = ,, 13·56

(*c*) Imaginary hole in top.—M. of I. = 63·47 – 2 holes $(\frac{3}{4} \times \frac{1}{2}) \times (3\frac{3}{4})^2$ = 52·92 in.^{4} and Z top and bottom = ,, 13·23

(The moments of inertia of the holes about their own axes are negligible.)

So that this method comes very close to the minimum Z found by the correct method of (*b*).

(*d*) Another current method is to deduct from the gross Z of 15·87 that of one hole (Fig. *b*), the value of which is area of hole × distance from the N.A. Thus, $15{\cdot}87 - (\frac{3}{4} \times \frac{1}{2}) \times 3\frac{3}{4}$ = in.^{3} 14·46
which figure is practically the average of the two Z's of (*b*).

(*e*) Still another approximate method, which, however, is more applicable to deep girders than to R.S.J's., is to take the stress, found by using the gross Z, and multiply it by the ratio of gross flange area to net flange area, *i.e.*, $\frac{\text{BM}}{\text{Z}} \times \frac{(4 \times \frac{1}{2})}{(3\frac{1}{4} \times \frac{1}{2})}$. This gives an equivalent modulus of Z gross × $(3\frac{1}{4} \div 4) = 15{\cdot}87 \times 3\frac{1}{4} \div 4$ = in.^{3} 12·9

Beam with Flange Plates.—Usually it is the bending moment or the deflection which settles the size of a joist or channel, and only occasionally the shear; the last occurs when the beam is of short span and is heavily loaded, or where the beam is purposely kept shallow to give head room, as in warehouse design.

In general, therefore, the most economical beam is that wherein most of the weight is concentrated in the flanges, which, without undue thinning of the web, should be as far apart as possible. Hence, it follows that the capacity of a beam to withstand bending moment can be greatly increased by the addition of flange plates.

Example.—To investigate whether the still shallower section of Fig. 185 could be used for the main beam of the floor already designed.

Gross M. of I, of $10'' \times 8'' \times 55$ lb. R.S.J.	$=$ in.^{4}	288·7
2 pls. $10'' \times \frac{3}{4}''$. M. of I. own axis (generally neglected)		
$= 2 \times \frac{1}{12}bd^3 = 2 \times \frac{1}{12} \times 10 \times 0{\cdot}75^3$	$=$,,	0·7
$+$ area $\times$ distance^2 $= 2\,(10 \times 0{\cdot}75)\,(5\frac{3}{8})^2$	$=$,,	433·4
Total gross M. of I.	$=$,,	722·8
$-$ inertia of 2 holes $= 2 \times$ area $\times$ dist.^{2}		
$= 2(1{\cdot}53 \times 0{\cdot}75) \times 5^2$ approx.	$=$,,	57·4
Total net M. of I.	$=$,,	665·4
Net section modulus $Z =$ net $I \div y = 665{\cdot}4 \div 5{\cdot}75$	$=$ in.^{3}	115·7
Web area given $= 10 \times 0{\cdot}4$	$=$ sq. in.	4

Against the requirements of 112 in.^{3} and 2 sq. in. respectively.

Obviously, however, the deflection rules this section out because the M. of I. is 723 against 1226 of the adopted 16-in. R.S.J., and so the deflection will be almost double. With the plated beam the deflection $= 0{\cdot}624''$ of the $16''$ joist $\times 1226 \div 723$ $=$ in. 1·06

This equals $1{\cdot}06 \div (25 \times 12)$, or $\frac{1}{283}$ of the span.

Deflection and Gross Areas.—The deflection is always calculated upon the gross moment of inertia, because when a structural element elongates, all the metal composing that element must elongate. Thus, consider a single flat bar suspender 4 in. wide and $\frac{1}{2}$ in. thick with a 1 in. diameter hole in it, then when it stretches all the metal whose gross area of cross-section is 2 sq. in. must lengthen and not only the metal on either side of the hole, the area of which metal is $1\frac{1}{2}$ sq. in. net.

Curtailment of Beam Flange Plates.—For the purpose of this paragraph assume the plated joist section to be satisfactory. When dealing with dead load it is common practice to assume that the dead load of a beam or plate web girder is concentrated at the

panel points, instead of being uniformly distributed. The we
of the 10″ × 8″ joist + two plates 10″ × $\frac{3}{4}$″ + rivet heads = 5
51 + say 3 = 109 lb. per foot run, giving a panel load of $\frac{1}{4}$

The total panel load = 4·5 + 0·25 = 4

The B.M. at A and F is zero ; at B and E =

$9{\cdot}5^{T} \times 5' = 47{\cdot}5$ ft. tons = in. tons 570

and between C and D it is constant at

$(9{\cdot}5^{T} \times 10' - 4{\cdot}75^{T} \times 5')$ ft. tons = ,, 855

From the B.M. diagram 186 it is seen that the rate of increas
B.M. between A and B is 570 ÷ 60″ = 9·5 in. tons per inch ru
beam. The 10 in. joist without flange plates has a Z of 57·74

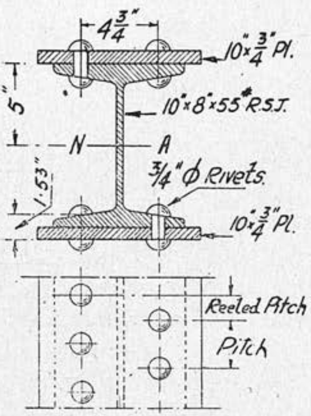


FIG. 185

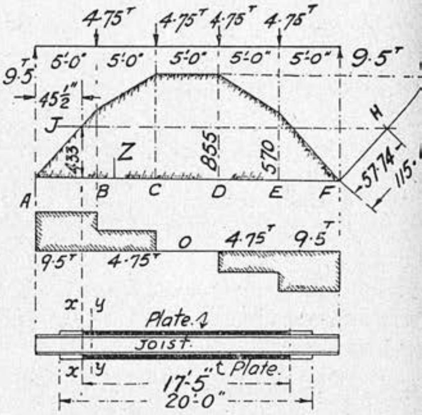


FIG. 186.

and the B.M. it can carry is Z × f_t = 57·74 × 7·5 = in. tons
This B.M. occurs at a distance of 433 ÷ 9·5 from
either end = in.

Between the column and this point the beam is strong enoug
carry the B.M. unaided ; the flange plates need extend, there
only between these two points, *i.e.*, a distance of 25′ − 2 enc
45½″ = 17′ 5″. The B.S.S. (4/11) specifies that these plate
carried past this theoretical point of cut-off for such a distanc
will contain a sufficient number of rivets to develop half the
strength.

The reasons underlying this rule are :—

(1) At the point of actual cut-off there is an instantaneous ch
of section modulus, but it does not follow that the distributio
stress will keep pace with the sudden change of section ; *e.g*
xx (Fig. 186) the B.M. is 433, joist Z 57·74 and flange extreme

stress 7·5 tons per square inch. At yy, say 2 in. to the right, the B.M. is still approximately 433, the Z is 115·4, while the extreme fibre stress on the plates $= 433 \div 115{\cdot}4 = 3{\cdot}75$ tons per square inch. If the latter stress exists, then the stress on the extreme fibres of the joist is proportional to their distance from the N.A., *i.e.*, $= 3{\cdot}75 \times \dfrac{5}{5{\cdot}75} = 3{\cdot}26$ tons per square inch. It is not conceivable that in such a short distance as xy the stress in the flange of the joist should drop from 7·5 down to 3·26 tons per square inch, and then gradually rise again to its maximum value at mid-span of $7{\cdot}5 \times \dfrac{5}{5{\cdot}75} = 6{\cdot}52$ tons per square inch. The change must be gradual, and must take place through the medium of the riveting. The additional length of plate permits of this diffusion of stress to take place before the point is reached where the plates are really required.

(2) The new specification is for bridges and the effect of rolling loads would influence its draughting. The wheel bases for concentrated loads, railway or road, vary, and with them their individual B.M. parabolas. The point of theoretical cut-off is probably fixed by one type of loading (assumed or actual), whereas another type, but little different, will have its B.M. parabola slightly to one side or other of the point of cut-off. The advantage of the additional length of plate is now apparent.

Number of End Rivets.

Plate strength $= (10'' \times \frac{3}{4}'' - 1 \text{ hole } \frac{3}{4}'' \times \frac{3}{4}'')$
$= \text{net area} = $ sq. in. 6·9
At 7·5 tons/sq. in. this represents $6{\cdot}9 \times 7{\cdot}5$ = tons 51·75
One $\frac{3}{4}$ in. diameter rivet in S.S., Table 12, Vol. III. = ,, 2·49
$\therefore$ Number of rivets to develop half plate =
$\frac{1}{2}\,(51{\cdot}75 \div 2{\cdot}49) =$ rivets 11
Using a $1\frac{1}{2}$ in. reeled pitch, extra length
per end $= 1\frac{1}{2}'' \times 11 =$ $1'\ 4\frac{1}{2}''$
Overall length of flange plate $= 17'\ 5'' +$
$2 \ @ \ 1'\ 4\frac{1}{2}'' = 20'\ 2''$, adopt $20'$

Allowing for the width of the columns the plates would probably be carried the full length of the joist in this particular case.

Table 10, Vol. III., gives a reeled pitch of 4·71 in. as being the minimum to adopt to prevent diagonal tearing, which, however, need not be investigated, there being a large excess of metal (as witness a flange stress of 3·75 tons/sq. in.) at this particular position. The rivet pitch should be kept small at the ends of the plate in order to develop the half strength of the plate in as small a distance as possible, and then increased as the centre line of the span is approached; see succeeding article on Rivet Pitch.

The calculation for finding the theoretical point of cut-off was considerably simplified by the fact that the B.M. diagram consists of straight lines. The following graphical method is much simpler and is extremely useful when the B.M. diagram is composed of curves. With point F as centre, swing a decimal scale round until FG registers 11·54. This may be on the eighths scale, or $\frac{1}{4}$ in. scale or even in centimetres, whichever scale works into the distances FG whose length is somewhat longer than the height of the B.M. diagram. The B.M. diagram, in turn, may be to any scale. In the original drawing, before reduction, the vertical or B.M. scale was $\frac{1}{8}'' = 100$ in. tons; linear scale (*i.e.*, length of beam AF) was $\frac{1}{8}'' = 1'\ 0''$; while FG was swung out on the decimal $\frac{1}{8}$ in. scale. The scales adopted are rather small for accurate work, but they suffice for purposes of illustration.

Along FG now prick off H, making FH = 5·774 on the scale of FG, representing the joist Z of 57·74 in.³. Through H draw a parallel HJ to the base line AF. Then so long as the B.M. diagram lies between AF and JH the joist modulus is sufficient; elsewhere the built-up section must be used. The point J is 433 in. ton units above AF as already found by calculation.

The Rivet Pitch of Plated Beams.—The formula used is $q = \frac{VG}{Ib}$ and is proved in Chapter I, Vol. II, where :—

q = the horizontal shear intensity in tons per square inch.

V = the total vertical shear, at the section considered, in tons.

I = the moment of inertia of the total gross section in inches⁴.

b = the gross breadth of the horizontal layer at the part considered.

G = the first moment (*i.e.*, gross area × distance) about the NA of all the area of metal above the horizontal layer considered.

Another common form for this formula is $q = \mathrm{S}ay \div \mathrm{I}b$, where S = V and ay = G.

Example.—To find the necessary pitching of the rivets for the foregoing plated beam. According to Fig. 186 the vertical shear V is constant at 9·5 tons between A and B. I = the gross M. of I. of 723 in.⁴ b = either the 8 in. width of the beam or the 10 in. width of the flange plates; it is immaterial which b is used, because in a 1 ft. length of the section the total force shearing the plates horizontally along the flanges of the joist is $q \times$ area per foot run $=$ $q \times b \times 12 = \frac{VG}{Ib} \times b \times 12 = \frac{12VG}{I}$, the term b cancelling out.

$G =$ area $\times$ distance $= (10'' \times \frac{3}{4}'') \times 5\frac{3}{8}''$ $=$ in.^{3} 40·3

Hence shear per horizontal foot

$$= \frac{12VG}{I} = \frac{12 \text{ in.} \times 9{\cdot}5^{T} \times 40{\cdot}3 \text{ in.}^3}{723 \text{ in.}^4} = \text{tons} \quad 6{\cdot}35$$

The rivets are in S.S. or bearing, and with $\frac{3}{4}$ in. diameter rivets the former value is the lesser and is $2{\cdot}49^{T}$.

Number of $\frac{3}{4}$ in. diameter rivets in 1 ft. length of either flange $= 6{\cdot}35 \div 2{\cdot}49 =$ 2·6

Reeled pitch in a 12 in. length of either flange $= 12 \div 2{\cdot}6 =$ in. 4·6

Any pitch not exceeding this withstands the shear, *e.g.*, a 4 in. pitch (reeled pitch adopted $= 1\frac{1}{2}''$).

Obviously $\frac{7}{8}$ in. diameter rivets would require to be pitched still wider and are, therefore, not used.

Portion BC.—With the floor fully loaded the shear is $9{\cdot}5^{T} - 4{\cdot}75^{T} =$ tons 4·75

This is not the maximum shear in the panel, which occurs when the portion FZ only is loaded with live load, as explained under. The shear is now $=$ tons 5·14

Shear per horizontal foot $= 12VG \div I =$ $12 \times 5{\cdot}14 \times 40{\cdot}3 \div 723 =$ tons 3·44

The reeled pitch works out at in. 8·7

(The reeled pitch adopted is $2\frac{1}{2}$ in.)

Portion CD.—The shear in this panel is zero when the floor is fully loaded. Maximum shear occurs when the live load covers half the span from either column up to the centre line. As the rivet pitch will be still wider than that of BC the shear will not be investigated further. (Reeled pitch adopted, 4 in.)

Actual Pitches Adopted.—The rivet pitch (*i.e.*, twice the reeled pitch) $\nless 2\frac{1}{4}$ in. and $\ngtr$ 9 in. ($3d$ and $12t$) (B.S.S. (4/25 and 26), and at the end of the plate $\ngtr$ 3·4 in., B.S.S. (4/11)). Adopt at the ends of the plate 3 in. pitch ($1\frac{1}{2}$ in. reeled pitch) for a length of 2 ft. 9 in. (B.S.S. (4/11)), *i.e.*, 3 in. past B; then from this point to C, nearly 5 ft., a pitch of 5 in.; and the mid panel CD a pitch of 8 in. The reeled pitches are, therefore, $1\frac{1}{2}$ in., $2\frac{1}{2}$ in. and 4 in., all well under the maximum pitches demanded by calculation; while the pitch of 8 in. in the mid panel permits the net calculated Z to remain as calculated, there being no possibility of diagonal tearing; see Table 10, Vol. III.

Maximum Shear in Panel BC.—The point Z has its positio
fixed such that BZ : ZC : : AZ : ZF or $\frac{BZ}{5 - BZ} = \frac{5 + BZ}{20 - BZ}$, henc
BZ = $1\frac{1}{4}$ ft. and ZC = $3\frac{3}{4}$ ft. This method applies to any pane
and is used here because it is much simpler than the usual formula
in text-books on the Theory of Structures. In words it is :—Let
divide the panel in the same ratio as it divides the span. In th
case of the centre panel, Z′ is at the centre line. Maximum positiv
shear is obtained by loading the right-hand portion FZ′ only, an
maximum negative shear by loading the left portion A to Z′. Th
latter is of no use in beam or plate web girder calculations, but i
in lattice girders.

Live load on beam per foot run @ 1 cwt/ sq. ft. of floor = 12·5 cwt. (Fig. 160) =	tons	0·625
Dead load per panel =	,,	1·625
With FZ loaded @ $0{\cdot}625^{T}$/ft. run, the live and dead load reactions at A are =	,,	4·4 and 3·25
Panel loads at B are, live and dead respectively =	,,	0·88 and 1·62
Hence maximum shear in the second panel $= (4{\cdot}4 + 3{\cdot}25)^{T} - (0{\cdot}88 + 1{\cdot}625)^{T} =$	,,	5·14

The point Z is known as the neutral point of the panel, because
load placed on the concrete floor at Z causes no live load shea
whatsoever in this panel. To verify this place a 1 ton load at Z
then $\frac{3}{4}$ ton comes to the B stringer and $\frac{1}{4}$ ton to the C stringer, an
thence into the main beam. The live load reaction at A, takin
moments at F, $= 1^{T} \times 18\frac{3}{4}' \div 25' = \frac{3}{4}^{T}$, and thence reaction
is $\frac{1}{4}^{T}$. Therefore, the live load shear in BC is reaction A − pane
load B $= \frac{3}{4}^{T} - \frac{3}{4}^{T} = 0$, as stated. Neutral points only exist wher
the load arrives at the main girder through stringers, *i.e.*, pane
loading.

Similarly, a 1 ton load placed on the concrete at mid-span causes
local load at C and D of $\frac{1}{2}^{T}$, and a reaction at A and F of $\frac{1}{2}^{T}$. Shea
from A to C is $\frac{1}{2}^{T}$ upwards, then at C the local $\frac{1}{2}$ ton load acts down
wards, leaving no vertical force or shear in panel CD.

FISH-PLATES AND SPLICES

Fish Plates.—Fig. 187, taken from a trader's hand book, is
typical illustration of a "standard" fish-plate. Fish-plates ar
useful for beam alignment, but in no way do they act as covers, a
the following example clearly indicates.

Beam Splices can be designed :—

(1) By using only the net areas of the various parts of the beam in the calculations.

(2) By using the moment of inertia and the shear area of the beam in the calculations.

(3) So that the splice plates and rivets are sufficient to carry only the actual B.M. and shearing force occurring at the section.

(1) *The Area Method* develops the total net tensile strength of the beam, and the calculations employing this method are given

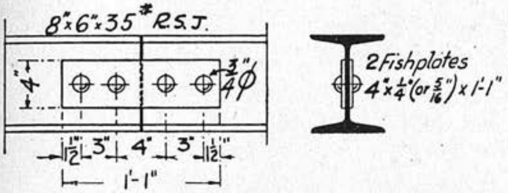


FIG. 187.

in the first few pages of Chapter VI., and refer to Fig. 188.

(2) *M. of I. Method.*

Gross M. of I. of beam (Fig. 188) $=$ in.^{4} 115·1

Minus 4 holes in flanges, section AA, $=$

$4(0{\cdot}648'' \times \frac{3}{4}'')\,(3{\cdot}676)^2 =$,, 26·1

Net M. of I. of beam $=$,, 89·0

The gross Z of the beam is listed at $=$ in.^{3} 28·8

The net Z of the beam $= 89{\cdot}0 \div 4''$ $=$,, 22·25

Covers :—

Gross M. of I. of flange covers $= 2 \text{ @ } 6'' \times \frac{11}{16}'' \times 4{\cdot}344^2 =$ in.^{4} 155·3

At AA, net M. of I. of flange covers $=$

$\frac{3}{4} \times 155{\cdot}3$, as the holes $= \frac{1}{4}$ width $=$,, 116·5

At AA, gross M. of I. of web covers $=$

$2 \text{ @ } \frac{1}{12} \cdot \frac{5}{16}'' \cdot 5\frac{1}{4}^3 =$,, 7·53

Total moment of inertia of covers at section AA $=$,, 124·03

$\therefore$ Net Z of covers at AA $= 124{\cdot}03 \div 4\frac{11}{16}''$ $=$ in.^{3} 26·5

Of the two sections of the beam alone, AA and BB, the former is the weaker, net flanges + gross web, and is, therefore, the actual strength of the beam which should be developed. The net modulus of the beam at CC equals that at AA, viz. a Z of a 22·25 in.^{3} net.

The unholed beam can carry a maximum B.M. of $Zf_t = 28{\cdot}8f_t$, but only a B.M. of $22{\cdot}25f_t$ when holed, so it follows that the splice should be situated at some position in the span where the B.M. does not exceed $22{\cdot}25f_t$.

If a pitch less than 3 in. be used the net area of the flanges will be

less than the gross area − 2 rivet holes each, because of diago
tearing (B.S.S. (4/4)).

Rivets :—

M. of I. of the rivets in the flanges = shear area × distance^2 = 2 @ $10 \times 0{\cdot}44 \times 4^2$, in terms of shear area = in.[4] 141

In terms of transverse area, since $f_s = \frac{3}{4} f_t = \frac{3}{4}$ of 141 = ,, 105

M. of I. of the rivets in the web = bearing area × distance^2 = 6 rivets @ $\frac{3}{4} \times 0{\cdot}35 \times 1\frac{3}{8}^2$, in terms of bearing area = ,, 3

In terms of transverse area, since $f_b = \frac{3}{2} f_t = \frac{3}{2}$ of 3 = ,, 4

Equivalent M. of I. of rivets ∴ = 105·75 + 4·5 = ,, 110

The net M. of I. of the beam, considering only minimum stren
was 89·0 in.[4] The net M. of I. of the covers = 124·03 and

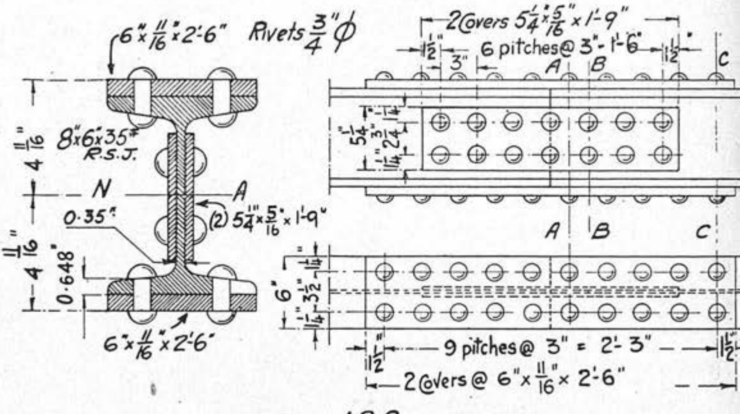


FIG 188

equivalent M. of I. of the rivets is 110·25, both in excess of the be
net M. of I.

The method of designing a splice "by areas" discusse
Chapter VI., and as illustrated by Fig. 188, is, therefore, ent
satisfactory when examined from the aspect of moment of iner

(3) *Actual B.M. and Shear Method.*—Suppose the splice be pla
as it usually is, at a section where the B.M. is small, *e.g.*, say
B.M. at the desired position is $Zf_t = 10f_t$, then there is no neces
to use heavy covers and riveting to develop a Z of 22·25, *i.*
B.M. of $22{\cdot}25f_t$. If it is permissible to curtail the flange p
when the B.M. decreases, surely the analogy holds good in that
the actually occurring B.M. should be developed, plus, say
allowance of 10 per cent. = a total of $11f_t$. Similarly, the

covers could develop the actual shear at the section plus 5 per cent. excess, since the web covers, unlike the flange covers, are symmetrical. Such a course is often adopted with plate girders, but a strict interpretation of the penultimate paragraph of the B.S.S. (4/22) would apparently exclude this last method from bridge design.

SPECIAL CASES OF BEAM LOADING

Bending in Two Planes. Approximate Solution, Fig. 189.—As a common everyday example consider the case of an ordinary side or end framing angle horizontal carrying galvanised sheeting. Reference to Figs. 157 and 158, Vol. II shows that the framing angle has to support two loads :—(1) The weight of the vertical sheeting which covers an area of 12 ft. 6 in. long by 6 ft. deep, *i.e.*, between horizontals.

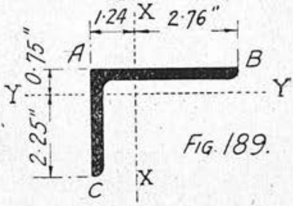


FIG. 189.

(2) The external wind pressure acting in a horizontal plane. Let the intensity of the wind pressure be 20 lb. per square foot of vertical surface.

Total weight of sheeting per angle of 12 ft. 6 in. span
= W, vertically, estimated at, tons 0·12

Total horizontal wind load P per angle
$= 12{\cdot}5' \times 6' \times 20$ lb./sq. ft. $\div 2240 =$,, 0·67

The minimum horizontal width of angle is limited to
$\frac{1}{40}$ span $= 150 \div 40 =$ in. 3·75

Either of two angles can be tried, (*a*) a $3\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$, or (*b*) a $4'' \times 3'' \times \frac{5}{16}''$ with the longer leg in the plane of greater B.M., *i.e.*, the horizontal one. The second angle, possessing a larger modulus for less weight of steel, appears to be the more economical.

M. of I., XX = 3·3.	Z left	$= 3{\cdot}3 \div 1{\cdot}24 =$	in.³	2·66.
	Z right	$= 3{\cdot}3 \div 2{\cdot}76 =$	,,	1·2
M. of I., YY = 1·59.	Z top	$= 1{\cdot}59 \div 0{\cdot}75 =$	,,	2·12.
	Z bottom	$= 1{\cdot}59 \div 2{\cdot}25 =$	,,	0·71

Assuming, meantime, that the ends of the angle are freely supported, then :—

B.M._W in vertical plane, due to W,
$= Wl \div 8 = 0{\cdot}12 \times 12{\cdot}5 \times 12 \div 8 =$ in. tons 2·25

B.M._P in horizontal plane, due to P,
$= Pl \div 8 = 0{\cdot}67 \times 12{\cdot}5 \times 12 \div 8 =$,, 12·56

W causes compression (+) on the AB face
and tension (−) at C;

while

P causes compression (+) on the AC face
and tension (−) at B.

Stresses at A.

$$\text{B.M.}_W \div Z_t = 2{\cdot}25 \div 2{\cdot}12 = +\ 1{\cdot}06$$
$$\text{B.M.}_P \div Z_l = 12{\cdot}56 \div 2{\cdot}66 = \underline{+\ 4{\cdot}72} = \text{tons/sq. in.}\ +\ 5{\cdot}7$$

Stresses at B.

$$\text{B.M.}_W \div Z_t = 2{\cdot}25 \div 2{\cdot}12 = +\ 1{\cdot}06$$
$$\text{B.M.}_P \div Z_r = 12{\cdot}56 \div 1{\cdot}2 = \underline{-\ 10{\cdot}46} = \quad ,, \quad -\ 9{\cdot}4$$

Stresses at C.

$$\text{B.M.}_W \div Z_b = 2{\cdot}25 \div 0{\cdot}71 = -\ 3{\cdot}17$$
$$\text{B.M.}_P \div Z_l = 12{\cdot}56 \div 2{\cdot}66 = \underline{+\ 4{\cdot}72} = \quad ,, \quad +\ 1{\cdot}5$$

The largest concentration of stress is fortunately a tensile one at B. The compression face AC is securely held by the vertica sheeting against any lateral movement in that plane.

Had the horizontal framing member been a joist, with the web horizontal, it would have had its principal axes parallel to both planes of loading, and the foregoing treatment would, except for the slight eccentricity of W acting on one face, have been free from reproach. Due, however, to the non-symmetry of the angle, the stresses found are very approximate.

Current Design of Framing Angles uses $Pl \div 10$ for the B.M. instead of $Pl. \div 8$ and entirely neglects the effects of the vertical dead load W. Experience has shown this to be quite satisfactory, and possibly the reasons for this are :—

(1) The sheeting is firmly bolted to the angles and forms with them one solid and massive section, wherein the sheeting serves as the compression flange of a "wind girder," and the outstanding angle legs act as ribs carrying tension. The moment of inertia of the combined elements must be much larger than that of the unaided angles.

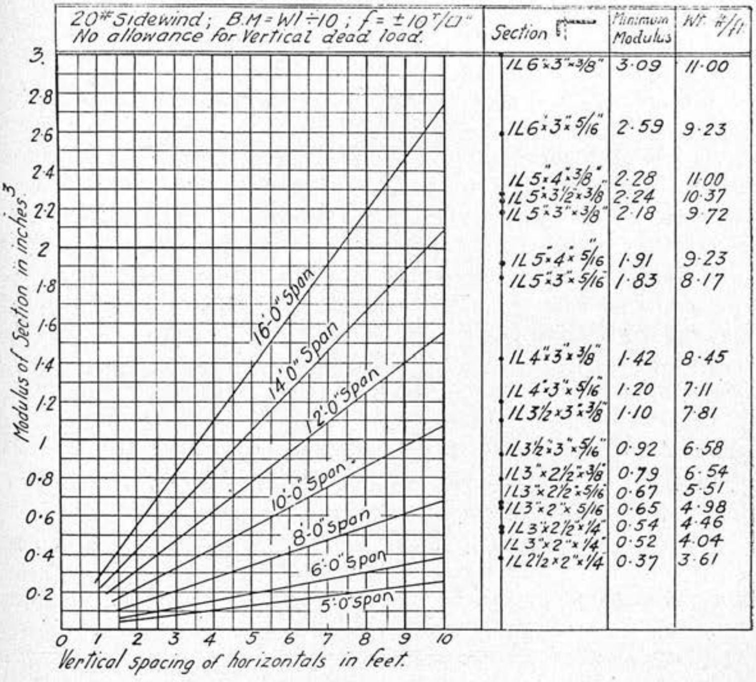
(2) With buildings of moderate height, say up to 40 ft. or thereby, the assumed wind pressure may never be reached owing to the frictional drag of the ground upon the wind gust.

(3) The combined sheeting probably acts as a very deep vertical girder, and is thus able to carry its own weight from column to column without stressing the horizontal angles, at least to any appreciable extent.

Although wind is a frequently occurring load, the maximum pressures are seldom reached; further, 20 lb. per square foot is equivalent to a wind velocity of almost 80 m.p.h. In view of this the working tensile stress for wind loads is commonly taken at 10 tons/sq. in., *i.e.*, an average factor of safety of 3. See also B.S.S. (3/3): 8^T/sq. in. + 25% = 10^T/sq. in. for dead + wind loads, etc.

On the foregoing assumptions, the stress at point B is as follows, using $Pl. \div 10$ for the B.M. due to wind and neglecting the dead load.

Max. B.M. due to wind	$= 0{\cdot}67 \times 12{\cdot}5 \times 12 \div 10 =$ in. tons	10·05
Wind stress at B	$= M \div Z = 10{\cdot}05 \div 1{\cdot}2 =$ T/sq. in. —	8·4



HORIZONTAL FRAMING ANGLES FOR SHEETING.

FIG. 190.

The graph given herewith (Fig. 190) is based upon current practice.

The immediately preceding remarks do not vitiate the example of bending in two planes, which is complete in itself.

Non-symmetrical Bending.—In all the examples upon the design of the floor beams the plane of loading coincided with one of the principal axes of the section and the neutral axis lay midway between the flanges. With the framing angle (Fig. 191), the horizontal wind pressure is parallel to YY, and therefore inclined to both the principal axes UU and VV.

The British Standards Association, in their complete lists of dimensions and properties of angles, give the following values for a $4'' \times 3'' \times 0{\cdot}3''$ angle. Area $= 2{\cdot}011$ sq. in., M. of $I_x = 3{\cdot}185$, $I_y = 1{\cdot}535$, $I_u = 3{\cdot}894$, $I_v = 0{\cdot}825$ and $\tan \alpha = 0{\cdot}548$ ($\alpha = 28{\cdot}7°$). Investigating only the action of the wind, the B.M. carried by the angle is 12·56 in. tons, *i.e.*, using $Wl \div 8$.

Resolve the B.M. into two components parallel to the principal

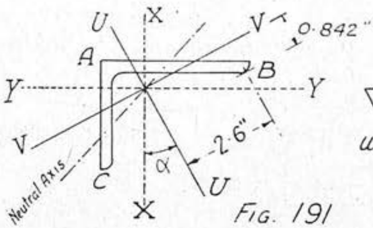


FIG. 191

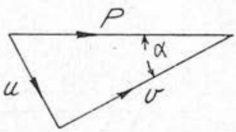


FIG. 192

axes UU and VV, and then find the fibre stress at B due to each considered separately, from the standard formula of $f = \dfrac{My}{I}$. The algebraic sum of these stresses gives the fibre stress at B. Refer to Fig. 192.

Component $U = P \sin \alpha = 12{\cdot}56 \times \sin 28{\cdot}7° =$ in. tons 6·03

Component $V = P \cos \alpha = 12{\cdot}56 \times \cos 28{\cdot}7° =$,, 11·02

About UU axis f (tension) $=$

$$\frac{My}{I} = \frac{Vy}{I_u} = \frac{11{\cdot}02 \times 2{\cdot}6}{3{\cdot}894} = \text{tons/sq. in.} - 7{\cdot}36$$

About VV axis f (tension) $=$

$$\frac{My}{I} = \frac{Uy}{I_v} = \frac{6{\cdot}03 \times 0{\cdot}842}{0{\cdot}825} = \quad ,, \quad - 6{\cdot}15$$

Total stress at B is tension $=$,, 13·51

The values I_u and I_v can be obtained from the abridged list of sections given in Table 4, Vol. III. Minimum radius of gyration $= 0{\cdot}64''$. $\therefore$ minimum M. of I. $=$ area $\times$ radius$^2 = 2{\cdot}01 \times 0{\cdot}64^2 = 0{\cdot}823$. Also I polar $= I_x + I_y = I_u + I_v$, hence $I_u = I_x + I_y - I_v = 3{\cdot}185 + 1{\cdot}535 - 0{\cdot}823 = 3{\cdot}89$ in.^{4}

It will be noted that a $4'' \times 3'' \times 0{\cdot}3''$ and not a $4'' \times 3'' \times \frac{5}{16}''$ ($0{\cdot}31''$) is considered; the difference in thickness of 0·01 in. may be neglected. The approximate method using the XX axis for the wind gave an extreme fibre stress of — 10·46 tons per square inch; an error of almost 23 per cent. Despite this large discrepancy, practice always uses the approximate method except for important members.

Stresses at the other corners of the angle can be found in a similar manner.

The foregoing problem can also be solved graphically by using the momental ellipse of which VV is the major and UU the minor axis. This method requires more time, and is not so accurate, as it entails the drawing, or at least part of, an ellipse; however, it is a useful check, and students are referred to text-books on Strength of Materials for the mode of procedure.

Three different sets of stresses for such a simple element of a structure as a framing angle are apt to be bewildering. The particular case of a framing angle was chosen to exemplify one of the many apparent divergences between theory and practice. Usually the reason for this difference hinges on the assumptions made by the designer; *e.g.*, either in the assumed value of the loads, in their distribution, or in the aid afforded by adjoining members. A brief *résumé* may clear the difficulty.

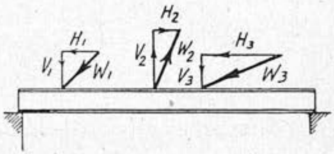


FIG. 193

(1) Bending in two planes.—The method is approximate when applied to a non-symmetrical section. In the case discussed it was assumed that the angle was free to deflect in any direction; an assumption which is incorrect.

(2) Bending in one plane, B.M. = $\overset{P}{\cancel{W}}l \div 10$ as given by the graph.—This method is empirical and indicates that practice recognises that the angle obtains aid from the sheeting and from the end cleats attaching the angle to the columns.

(3) The method given under the heading of non-symmetrical bending is the correct one to employ in place of (1) when the section is non-symmetrical. As with the approximate method of (1) the angle was assumed free to deflect in any direction, an assumption which has a large influence upon the resulting calculated stresses.

The Load Inclined to the Neutral Axis but Acting in the Plane of the Web of the Beam. Fig. 193 illustrates a girder with three such inclined loads, W_1, W_2 and W_3.

Method.—Resolve the inclined loads into their vertical and horizontal components and treat the problem after the method given in Chap. IV, Vol. II on "Combined Direct and Lateral Thrust."

The Load Inclined to the Plane of the Web.

Fig. 194 shows the nose of a plated cantilever girder in the gallery of a theatre. The load W of Fig. (*a*) may, with small error, be assumed to pass through the centre of gravity of the joist. Now resolve either W or, more simply, the vertical bending moment M, occasioned by W, into two directions parallel to the principal axes XX and YY. The component M cos θ creates compressive stresses

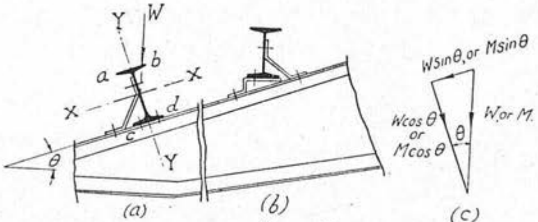


FIG 194

along the *ab* face and tensile stresses along *cd*. Similarly, M sin θ produces compression at *b* and *d* and tension at *a* and *c*.

$$\begin{aligned} \therefore\ \text{Stress at } a &= +\mathrm{M}\cos\theta \div \mathrm{Z}_x, -\mathrm{M}\sin\theta \div \mathrm{Z}_y. \\ \text{Stress at } b &= +\mathrm{M}\cos\theta \div \mathrm{Z}_x, +\mathrm{M}\sin\theta \div \mathrm{Z}_y. \\ \text{Stress at } c &= -\mathrm{M}\cos\theta \div \mathrm{Z}_x, -\mathrm{M}\sin\theta \div \mathrm{Z}_y. \\ \text{Stress at } d &= -\mathrm{M}\cos\theta \div \mathrm{Z}_x, +\mathrm{M}\sin\theta \div \mathrm{Z}_y. \end{aligned}$$

Maximal stresses occur at *b* and *c*.

Detail *b* eliminates the non-symmetrical bending by using a $\frac{5}{8}$ in. thick, or thereby, M.S. bent plate or chair. The bent plate cleat to the web serves as a safeguard against overturning.

CHAPTER XI

DESIGN OF A JOIST AND CHANNEL CRANE GANTRY GIRDER

In modern shop equipment much depends upon the crane installation. Rapidity of handling loads is essential towards shop output, and crane design has kept pace with the demands made upon it.

The steelwork which carries these quick-acting cranes must be heavier than the steelwork which supports slow-moving cranes. With quick-acting electric overhead travelling ("E.O.T.") cranes the stresses in the gantry girders are produced almost instantaneously, whereas with slow-moving hand-operated cranes the bending stresses in the girder are induced gradually from zero up to their maximal values, as the crane traverses the girder from the end towards the centre.

To allow for this, and taking into consideration that f_t is taken at 8^{T}/sq. in., an impact factor varying from 25 per cent. to 30 per cent. should be added to all the live load stresses caused by E.O.T. cranes. For hand-operated cranes an increase of $12\frac{1}{2}$ per cent. on the live load stresses should be ample. Stresses due to speeding and braking, and also crane mismanagement, are more liable to occur in a long, wide workshop than in one which is cramped, and thus the 30 per cent. impact factor should be reserved for the longer gantry. Should the cranes be working in a heavy foundry where loads up to (or occasionally over) the full lift capacity are continually encountered it might be advisable to raise the impact allowance to 35 per cent., but for ordinary shops where the loads vary from light to heavy an impact addition of 25 per cent. to 30 per cent. appears to be quite reasonable and in consonance with good practice. The impact formula of the B.S.S. for bridges cannot be used here, as the speeds and type of loading are entirely different.

Permissible or working stresses vary somewhat with designers. Some firms in designing crane girders use a working tensile stress of 6 to 6·5 $^{\text{T}}$/sq. in. for the combined dead and live load stresses without any impact factor. Where the dead load stresses are accurately known there is no apparent reason why the working stress should not be 8 $^{\text{T}}$/sq. in. as for bridge work. The live loads could then be associated with impact factors which would vary with the nature of the loading. However, it is not necessary to enter into polemics on this subject. In the following example f_t will be taken at 8^T/sq. in. in conjunction with an impact allowance of 30 per cent.

Comparing the two above working stresses for a tensile live load flange stress of W^T :—

Method adopted.

Required area $= (W + 30\%) \div 8 = 0{\cdot}163W$ net sq. in.

Other method.

Required area $= W \div (\text{from } 6 \text{ to } 6{\cdot}5) = 0{\cdot}167W$ to $0{\cdot}154W$ net sq. in.

Longitudinal Forces on Crane Girders Fig. 195.—The largest of these, especially in quick-acting electric overhead travelling cranes, is due to the sudden application of the brakes. The frictional resistance to the sliding of the locked wheels upon the rail is supplied by the crane girder. This element in turn distributes it amongst all the crane column shafts.

The coefficient of friction μ for steel sliding on steel is variously stated at 0·14 to about 0·2 (see the B.S.S. (3/9 and (3/26)). Thus the longitudinal forces on the gantry track of two lines of girders may be assumed as never exceeding one-fifth of the total weight of the crane and its lift load.

As a particular example :—A large-span E.O.T. crane of 40 tons lift weighs 32 tons without its load. The total load carried by the gantry girders is 72 tons, and hence the total maximum longitudinal thrust along the girders is 0·14 to 0·2 of 72, *i.e.*, from 10 tons to 14·4 tons ; or, assuming each girder takes an equal share, 5 tons to 7·2 tons per girder.

The above implies that the load is in mid-shop, whereas if the load lifted be adjacent to one side of the building, that pair of wheels nearer the load will carry 28 tons each, depending upon the width of shop, instead of 18 tons each ($72 \div 4$ wheels), and the longitudinal thrust on this girder, theoretically, may rise to 2 (wheels) $\times$ $28 \times \mu$, *i.e.*, 11·2 tons.

This force will be carried from the rail surface through the rivets or bolts connecting the rail to the top flange. These are usually spaced at a reeled pitch of about 1 ft., giving a total of 41 or thereby in a 40-ft. span gantry girder. The shear load per fastening is $11{\cdot}2 \div 41 = 0{\cdot}27$ ton, so that these fastenings are safe. No further notice is taken of this thrust in designing the crane girders ; but it should be taken into consideration when designing the columns and other parts of the structure.

Consider this question from another aspect. Say the speed of longitudinal or main travel is 250 ft. per minute and that the crane can brake itself in a length of 10 ft. To find the retarding force supplied by the gantry girder :—

$v^2 = u^2 + 2as$. Where v = final velocity in feet per second = 0

$\therefore 0 = (4\frac{1}{6})^2 - 2a \times 10$ $\quad u$ = initial velocity in feet per second = 250 ft. per min. = $4\frac{1}{6}$ ft. per sec.

whence $a = 0{\cdot}87$

a = acceleration in feet per second per second ; when braking it is retardation or negative.

s = space travelled with brakes on before coming to rest = 10 ft.

Force = mass × acceleration. Assuming that the retarding force is constant (which it is not) over this 10-ft. length, then Force $= \dfrac{72 \times 2240 \times 0{\cdot}87}{g}$ lb., or in tons $= \dfrac{72 \times 0{\cdot}87}{32{\cdot}2} = 2^{\mathrm{T}}$ nearly.

This force of 2 tons is the average force acting on the pair of gantry girders. The actual maximum force probably lies nearer the first figure found by the frictional method. Specifications naturally reflect this wide divergence of values. Thus the horizontal longitudinal thrust due to braking is given in one specification as one-eighth of the total load on the wheels, and as one-twentieth by another.

Rolling friction will cause also a longitudinal thrust through the main gantry track, but its value is much smaller than the braking thrust.

Experimental results on friction are highly conflicting and the difficulty of assessing thrusts will be seen from the following table. Table A contains the results of Captain Galton and Mr. Westinghouse, and are for steel wheels sliding on steel rails. M. Poirée, experimenting on the Paris and Lyons Railroad with a four-wheeled waggon, whose brakes were screwed up so that the wheels skidded, found the results given in Table B.

A		B	
Speed in miles/hour.	μ	Speed in miles/hour.	μ
10	0.110	9—14	0.208
15	0.087	14—18	0.179
25	0.080	18—20	0.167
38	0.047	30—40	0.144

Although both agree that the coefficient decreases with an increase of speed, its values as found by M. Poirée are twice those as found by Messrs. Galton and Westinghouse.

Lateral Forces on Crane Girders (Fig. 195) may be caused by :—

(*a*) The thrust due to the sudden stopping of the crab and load when traversing the crab girders.

(*b*) The crane dragging weights across the shop floor.

The foregoing cases cannot occur simultaneously.

In (*a*), as with the longitudinal gantry girders, the frictional resistance of the rail is transferred into the crab girders and from them into the crosshead girders, thence, as point loads through the main wheels, into the top or compression flanges of the gantry girders.

The positions of the main wheels when maximum lateral bending

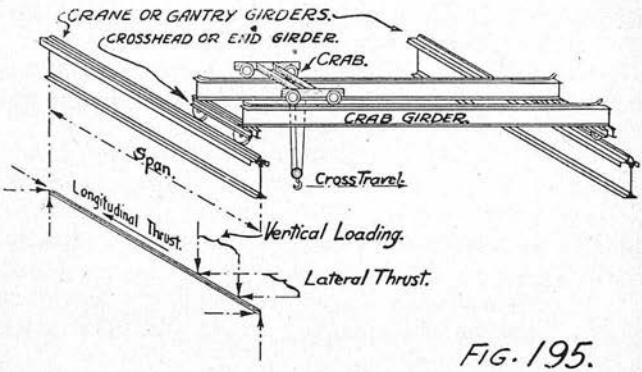


FIG. 195.

and shear take place on the gantry girder will be the same as those when maximum vertical bending moment and shear occur.

It is just as difficult to determine the value of this lateral thrust as it was to fix a value for the longitudinal thrust. The following table is a summary of what various authorities suggest as being a proper allowance to make for lateral thrust.

c = weight of crab + ropes and W = weight lifted.

A.	$\frac{1}{7}$ (W + c)	Treated as a dead load.
B.	$\frac{1}{10}$ (W + c)	Treated as a live load (*i.e.*, impact has to be added to this figure.
C.	$\frac{1}{10}$ (W)	Treated as a live load (*i.e.*, impact has to be added to this figure).
D.	$\frac{1}{5}$ (W)	Treated as a live load (*i.e.*, impact has to be added to this figure).

As the weight of the crab is roughly proportional to the load lifted (from one-third to one-fifth), the allowance for lateral thrust can be expressed wholly in terms of the load capacity W of the crane. The average of the above summary works out at approximately $\frac{1}{7}$W treated as a live load. The equivalent static or dead load would be $\frac{1}{7}W \times \frac{130}{100}$, *i.e.*, adding 30 per cent. for impact. This force can be assumed as being carried equally by all the wheels on the main gantry track. If there are two wheels per track then each wheel would exert a horizontal thrust on the compression flange of $\frac{1}{4}\left(\frac{W}{7} \times \frac{130}{100}\right)$.

(*b*) The crane is often requisitioned to drag weights across the shop floor. If the load is extremely massive it is usually mounted on roughly fashioned rollers, probably running on a timber plank track. The lateral thrust and pull on the compressive flanges of the gantry girders then become a matter of conjecture. The resisting forces are, firstly, the friction of the main wheel treads upon the gantry rails and, secondly, the forces offered by the flanges of the main wheels bearing against the gantry rails.

The suggested figure of $\frac{W}{7}$ + impact for lateral thrust should amply cover for both cases (*a*) and (*b*), as practice very often makes no allowance for thrust beyond building the upper flanges of the gantry girders extremely wide and of the inverted U type. Many again make no allowance whatsoever; an example from this type is quoted under.

Flange Width is usually not less than a twenty-fifth of the span. The upper flange is a compression member and may fail sideways, especially under side or lateral thrust. The outer edges of the flanges are stiffened by having a continuous angle riveted to them or, if the crane is light, the upper flange is composed of an inverted channel as in Fig. 200. Using the deepest channel rolled, viz., 17″ × 4″ × 44 lb., the limiting span for this type is in the neighbourhood of 35 ft. The narrower the flange width is in proportion to the span the smaller is the working stress (B.S.S. (3/18)). When the flange plates are not stiffened by edge angles the working stress is further lowered.

A 5 tons E.O.T. crane has been running for several years now on gantry girders of 20″ × 7½″ × 89 lb. R. S. Joists. The unsupported span is 32 ft., so that the ratio of flange breadth to span works out at the alarming figure of $\frac{1}{51}$.

EXPLANATORY TEXT

DESIGN OF A 20-FT. SPAN GANTRY GIRDER

Dead Load, item **1**.—The live load bending moment was calculat as in item **4**, and—neglecting dead load meantime—this divided the working tensile stress of 8 tons per square inch gives an appro mation to the required modulus. Thus $673 \div 8 =$ a modulus about 84. The rules of items **9** and **10** were then considered, fro which was evolved the section of Fig. 200. Allowing for a rail 45 lb. per yard, the dead weight works out at 0·8 of a ton.

Crane Rails.—A railway rail supported on sleepers—spaced 2 ft. 6 in. centres or thereby—is permitted to carry a wheel load 1 ton for every 10 lb. per yard weight of rail. On this basis t Pacific Type of Locomotive, which has a 10 tons wheel load, wou require a 100 lb. per yard rail. Crane rails, on the other hand, a supported throughout their entire length ; there is no bridge beam action between sleepers, and in consequence the rail may permitted to carry a heavier running load. Practice would app rently assign a safe wheel load of 3 tons for every 10 lb. per yard "bridge rail," and for special "flat-bottomed" rails a wheel lo of $3\frac{1}{2}$ tons to 4 tons per 10 lb. per yard of rail.

The rail is assumed to offer no help to the crane girder in carryi the loads.

Item **2**.—The reaction will be a maximum just previous to t leading wheel passing off the span. Live load reaction A is fou by taking moments about end B.

Item **4** *and Fig.* 197.—"Maximum bending moment, under a wheel, occurs when the centre line of the span is midway betwe the centre of gravity (C.G.) of the load system and that whee Both wheels have the same load and their joint C.G. therefore l midway between them. The position for maximum B.M. th occurs when the centre line (C.L.) of the span lies between the whe and is at a distance of a quarter of the wheel base from either whe Live load reaction B is found by taking moments about A.

If the wheel base is greater than 0·586 the effective span maximu B.M. will occur when one wheel is off the span and the remaini wheel at mid-span.

Maximum Dead Load B.M. takes place at mid-span and is equ to $Wl \div 8$. The B.M. parabola changes its shape but slightly the near neighbourhood of the centre line, so that the B.M. at X approximately that at mid-span. The error of this assumption small and is on the right or full side.

Items **6**, **7** *and* **8**.—Fig 196 gives the position of the wheels maximum horizontal shear (due to lateral thrust) as well as t position of loads for maximum vertical shear. The distances are the same, the only difference being in the planes of loading and values of the loads. Hence item **7** is obtained directly from item **3** by multiplying the latter by the ratio of the loads, viz., 0·27 ÷ 8. Similarly, item **8** = item **4** × (0·27 ÷ 8).

CALCULATIONS

DESIGN OF A 20-FT. SPAN GANTRY GIRDER (FIG. 200)

Loading, etc.—One E.O.T. crane of 7·5 tons capacity with a 10-ft. wheel base. Maximum load on each end carriage wheel is 8 tons, which occurs when the crab and the load are adjacent to the main

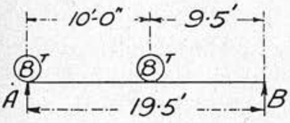


FIG. 196.

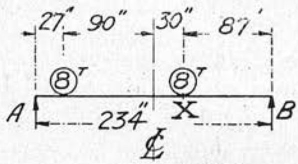


FIG. 197

gantry rails. Permissible tensile stress = 8 tons per net square inch. Impact = 30 per cent.

Effective Span.—Allowing that the girder has a 6 in. length of bearing on each column bracket support, the effective span is 19 ft. 6 in., *i.e.*, centre of bearing to centre of bearing.

Dead Load of Girder and rail is estimated at ton 0·8 **1**

Maximum Vertical Shear.—Fig. 196.

Live load reaction A =

$8^{T}(9{\cdot}5' + 19{\cdot}5') \div 19{\cdot}5 =$ tons 11·9

Impact, add 30% = ,, 3·6

Total L.L. + I. = ,, 15·5 **2**

Dead load reaction A = 0·8 ÷ 2 = ,, 0·4

Total end shear L.L.+I.+D.L. = tons 15·9 **3**

Maximum Vertical B.M.—Fig. 197.

Live load reaction B =

$8^{T}(27 + 147) \div 234 = 5{\cdot}95$ tons.

Maximum B.M. @ X, L.L. =

$R_B \times 87'' = 5{\cdot}95 \times 87 =$ in. tons 518

Impact, add 30% = ,, 155

Total L.L. + I. = ,, 673 **4**

Dead load maximum B.M. @

mid-span = $0{\cdot}8^{T} \times 234 \div 8 =$,, 23

Total maximum B.M. @ X is

approximately = in. tons 696 **5**

Lateral Force = $\frac{1}{7}$ of
lift capacity = $7{\cdot}5 \div 7$ = tons 1·07
per wheel = $1{\cdot}07 \div 4$ wheels = ton 0·27 6
Lateral Maximum Shear, includ-
ing 30% impact = $15{\cdot}5 \times \dfrac{0{\cdot}27}{8{\cdot}0}$ = ,, 0·5 7
Lateral Maximum B.M., includ-
ing 30% impact = $673 \times \dfrac{0{\cdot}27}{8{\cdot}0}$ = in. tons 22·7 8

EXPLANATORY TEXT

Position of N.A.—The centroid of the channel's cross-sectional area is given in the list of properties of sections as 0·78 in. from the heel. Then the channel area multiplied by the distance of its centroid from the base, plus the R.S.J. area multiplied by its distance of centroid, gives the total moment of area about the base. This figure divided by the total area gives the position of the joint centre of gravity and, therefore, of the neutral axis. No deductions are necessary on account of rivet holes, as these occur in the compression flange. The rivet fills the hole, and so is assumed to carry the thrust or compression in the top flange fibres from one side of the hole to the other.

Moment of Inertia, or more correctly, second moment, about any line parallel to that through the centroid of the area = M. of I. about the centroidal line plus the area of the section multiplied by the square of the distance between the parallel lines of reference, *i.e.*, M. of I_1 = M. of $I_0 + Ad^2$ as given in text-books on Mechanics.

It is immaterial whether the centroid of the component part be above or below the neutral axis of the combined section, M. of I. is always the sum. Algebraically d may be positive or negative with reference to the xx axis, but its square is always positive. The inertias and areas of the sections are taken from the tabulated list of properties of sections.

Item **13**.—The lateral thrust is assumed to act in the plane of the centre of gravity of the upper flange. Acting as it does at rail level, it has really a lever arm producing torque. This small lever arm and, therefore, the torque are neglected. No help is assumed to be afforded by the lower or tensile flange in resisting lateral thrust. However, should this help be considered, then the torque—due to the thrust multiplied by the distance from the line of action of the thrust to the N.A. (xx in this case)—should also be taken into consideration.

Item **14**.—Section modulus = M. of I. ÷ distance from N.A. to the fibre considered and is in inches^3, (inches^4 ÷ inches). The outermost fibres have the greatest stress, and since these are situated at unequal distances from the N.A. there will be a modulus for the top fibres and another for the bottom fibres. It appears rather puerile to give the foregoing, but inertias and moduli always provide a trap for beginners.

CALCULATIONS

Girder depth will be from span ÷ 10 to span ÷ 12, *i.e.*, from in. 23 to 19 **9**

Width of top flange will be about span ÷ 25 = ,, 9 **10**

Approximation to required modulus = B.M. ÷ f_t = 696 ÷ 8 = in.³ 87 **11**

The foregoing three items tentatively fix the section as being composed of an 18 in. deep R.S.J. (modulus 93·3) with a 9-in. channel riveted to the upper flange.

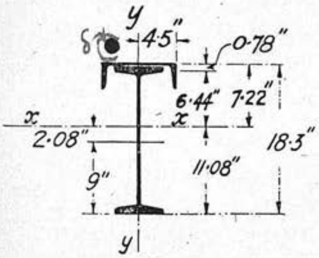


FIG 198.

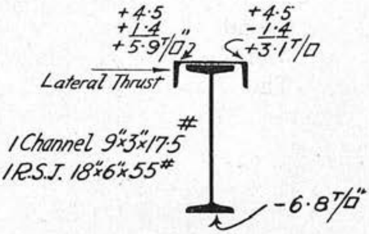


FIG. 199.

Position of Neutral Axis. Moments about base.

·18″ × 6″ × 55 lb.
R.S.J. 16·18 sq. in. × 9″ = in.³ 145·62

9″ × 3″ × 17·5 lb.
Channel 5·14 (18·3—0·78) = ,, 90·75
21·32 sq. in. ,, 236·37

N.A. is situated above the base at a distance of 236·37 ÷ 21·32 = in. 11·08

Moment of Inertia xx axis.
R.S.J. own axis = in.⁴ 841·8
R.S.J. Area × distance² = 16·18 × 2·08² = ,, 69·9
Channel. Own axis (*i.e.*, the *yy* axis of Table 5, Vol. III.) = ,, 3·75
Channel. Area × distance² = 5·14 × 6·44² = ,, 213·16

Total moment of inertia *xx* axis = in.⁴ 1128·6 **12**

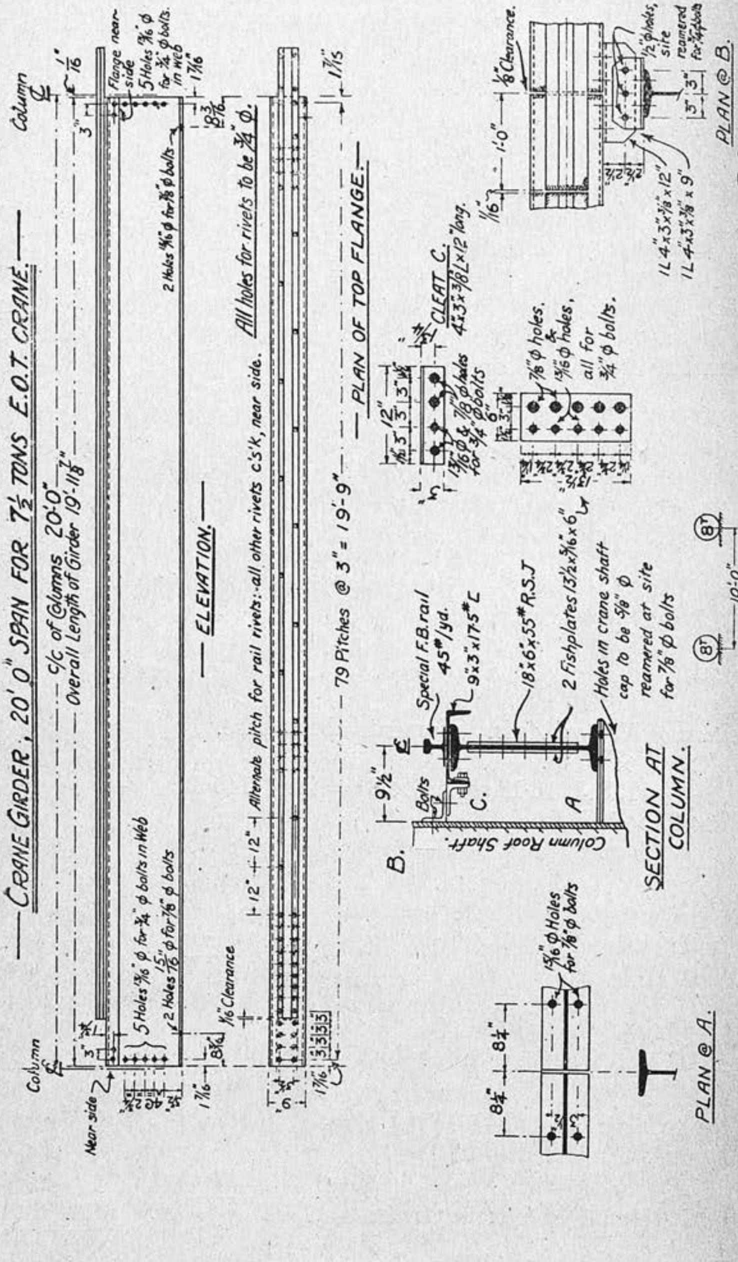

CRANE GIRDER, 20' 0" SPAN FOR 7½ TONS E.O.T. CRANE.
c/c of Columns 20'0"
Overall Length of Girder 19'-11 7/8"
Column ℄
Near side
Flange near side
ELEVATION.
1/16" Clearance
Alternate pitch for rail rivets:-all other rivets c's'k, near side.
All holes for rivets to be ¾" φ.
79 Pitches @ 3" = 19'-9"
PLAN OF TOP FLANGE.
Special F.B. rail 45#/yd.
9"x3"x17.5# C
18"x6"x55# R.S.J
Holes in crane shaft cap to be 5/8" φ reamered at site for 7/8" φ bolts
Column Roof Shaft.
Bolts
SECTION AT COLUMN.
CLEAT C.
7/8" φ holes.
all for ¾" φ bolts.
1/8" Clearance.
PLAN @ A.
PLAN @ B.
LOADING.
10'-0"

FIG. 200.

Moment of Inertia yy axis top flange.

Channel. Own axis (*i.e.*, the xx axis of Table 5, Vol. III.) $=$ in.^{4} 62·52

R.S.J. top flange only $= \frac{1}{2}$ tabular yy inertia, approx. $= 23{\cdot}64 \div 2 =$,, 11·82

Total M. of I. of upper flange, yy axis $=$ in.^{4} 74·3 **13**

Moduli of Section.

Zxx upper $= I_{xx} \div 7{\cdot}22 = 1{,}128{\cdot}6 \div 7{\cdot}22 =$ in.^{3} 156 **14**

Zxx lower $= I_{xx} \div 11{\cdot}08 = 1{,}128{\cdot}6 \div 11{\cdot}08 =$,, 102 **15**

$Zyy = I_{yy} \div 4{\cdot}5 = 74{\cdot}3 \div 4{\cdot}5 =$,, 16·5 **16**

Working Stresses.—B.S.S. (3/18).

Tension $=$ T/sq. in. 8 **17**

$$\text{Compression } 8\left(1 - 0{\cdot}0075\,\frac{l}{b}\right) =$$

$$8\left(1 - 0{\cdot}0075\,\frac{20' \times 12}{9}\right) = \quad ,, \quad 6{\cdot}4 \quad \mathbf{17}$$

EXPLANATORY TEXT

Items **18** to **20**.—Due to vertical loading, the upper flange is in compression with the extreme top lamina of 9 in. width stressed to 4·5 tons per square inch. In addition, assuming that the lateral thrust comes from the left, the horizontal loading causes further compression in the left channel leg, while the right channel leg tends to be in tension; the upper flange of the girder deflecting in a horizontal plane, concave side towards the left hand and convex side towards the right. The stresses are added together as shown on Fig. 199.

Item **21** would apparently indicate that the section is rather heavy. On looking at the list of joists it will be seen that the next lightest joist is $16'' \times 6'' \times 50$ lb., a saving of 5 lb. per foot over the 18-in. joist. The ratio of depth to span, however, falls from $\frac{1}{13}$ to $\frac{1}{14{\cdot}5}$, and therefore deflection will be larger with the lighter joist. If the 9-in. channel be replaced by an 8-in. channel the ratio of flange width to span drops from $\frac{1}{26}$ to $\frac{1}{30}$, with a consequent lowering of the working compressive stress (item **17**). Further, from Table 13, the 9-in. channel possesses a clear distance between root fillets of 7 in., which is ample for the 6-in. flange of the joist; on the other hand, the $8'' \times 3''$ channel has 6 in. exactly, and therefore no allowance is given for sectional growth of the joist flange.

The calculated section will therefore be used. Details are given in Fig. 200.

Item **23**.—The breadth of section b may be taken as 6 in., the flange width of the joist, or the channel width of 9 in.; it is immaterial which value is adopted as the breadth cancels out in item **24**.

Fish-plates are used for the purpose of alignment, the thickness shown on the drawing being adopted without calculation. When fitting these, the $\frac{13}{16}$ in. diameter holes are placed, say, at the right end of girder and the $\frac{7}{8}$ in. diameter holes at the left end. The latter are equivalent to, but cheaper than, slotted holes. They permit of a longitudinal adjustment of the girder, as also does the $\frac{1}{8}$ in. clearance between the ends of the girders.

Transverse adjustment is obtained by reamering out the holes in the crane column cap and also the common holes of the column cleat connection of detail B. The crane girders must be exactly parallel and true to line before being permanently bolted up to the main structure. The $9\frac{1}{2}$ in. from the rail centre line to the face of the roof shaft is net; should there be rivet heads on the roof shaft, then the extra $\frac{1}{2}$ in. or more must be added to the $9\frac{1}{2}$ in. dimension.

Temperature.—The expansion due to temperature is seldom provided for in this country with crane girders. If expansion becomes large the girders will move the bolts in the "slotted" fish-plates.

Rail joint at 1 ft. from the end of the girder causes less jarring than if it were at mid-span, or even at the column centre line where the crane girders butt end to end.

CALCULATIONS

Actual Stresses.—Vertical loading.

$$\text{Tension} = \text{B.M.} \div Z = 696 \div 102 = -6{\cdot}8\ {}^{\text{T}}/\text{sq. in.} \qquad \mathbf{18}$$

$$\text{Compression} = \text{B.M.} \div Z = 696 \div 156 = +4{\cdot}5\ {}^{\text{T}}/\text{sq. in.} \qquad \mathbf{19}$$

Horizontal loading

$$\text{Tension and compression} = 22{\cdot}7 \div 16{\cdot}5 = \pm 1{\cdot}4\ {}^{\text{T}}/\text{sq. in.} \qquad \mathbf{20}$$

Resulting stress in top flange,

$$\text{extreme fibres} = +4{\cdot}5 \pm 1{\cdot}4 = \begin{cases} +5{\cdot}9\ {}^{\text{T}}/\text{sq. in.} \\ +3{\cdot}1\ {}^{\text{T}}/\text{sq. in.} \end{cases} \qquad \mathbf{21}$$

See Fig. 199.

The maximal stresses of $-6{\cdot}8^{\text{T}}$ and $+5{\cdot}9^{\text{T}}$ are under the permissible stresses of -8^{T} and $+6{\cdot}4^{\text{T}}$ for tension and compression respectively.

Shear.—Area of joist carrying vertical shear by the B.S.S. (4/4) is web thickness × overall

$$\text{depth of joist} = 0{\cdot}42'' \times 18'' = \text{sq. in.} \quad 7{\cdot}56$$

Resulting vertical shear
stress at end $= 15{\cdot}9 \div 7{\cdot}56 =$ T/sq. in. 2·1 **22**
Resulting horizontal shear stress at end is
$0{\cdot}5^T \div$ area of channel web $=$ negligible.
Working shear stress on webs (B.S.S. (3/18)) $=$ T/sq. in. 5

Riveting.—The intensity "q" of the longitudinal shear between [jo]ist flange and channel, where they have a common area of contact, [is] given by the formula

$= \frac{VG}{Ib}$ Where V $=$ vertical shear at the point $= 15{\cdot}9$, item **3**.

I $=$ moment of inertia at the section $= 1128{\cdot}6$, item **12**.

G $=$ first moment of the area above the plane considered $= 5{\cdot}14 \times 6{\cdot}44$.

b $=$ breadth of section at the point considered $=$ width of joist flange $= 6''$

[R]iveting will be closest at the end of span.

$$q = \frac{15{\cdot}9 \times 5{\cdot}14 \times 6{\cdot}44}{1128{\cdot}6 \times 6} = \text{T/sq. in.} \quad 0{\cdot}078 \quad \mathbf{23}$$

[∴] Total shear per inch run of span for a 6 in. width of flange $= 0{\cdot}078 \times 6 =$ $0{\cdot}468^T$ **24**
Total shear per foot run of span for a 6 in. width of flange $= 0{\cdot}468 \times 12 =$ $5{\cdot}616^T$
[U]sing $\frac{3}{4}$ in. diameter rivets.
Value of one in single shear $= 0{\cdot}44 \times 6 =$ $2{\cdot}64^T$
and in bearing (B.S.S. (3/18)) on channel web $= \frac{3}{4}'' \times 0{\cdot}3 \times 12 =$ $2{\cdot}7^T$
[N]umber of rivets required per foot $= 5{\cdot}616^T \div 2{\cdot}64^T =$ 3
[M]aximum pitch for compression member rivets $= 12t$ (B.S.S. (4/26)) $= 12 \times 0{\cdot}3 =$ in. 3·6
[A]dopt 3 in. pitch throughout, see Fig. 200).
Number of rivets per foot $=$ 8

Stiffeners.

[R]atio of web thickness to joist depth $= 0{\cdot}42 \div 18 = \frac{1}{43}$

No stiffeners required, since these are only necessary (B.S.S. (3/12)) when the ratio is less than $\frac{1}{60}$.

INDEX

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